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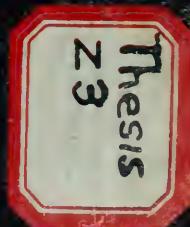
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AN INVESTIGATION OF PRESTRESSED
CONCRETE STRUCTURES

ALEXANDER PHILIP ZECHELLA
AND
BRYAN SEVERANCE PICKETT



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Annapolis, Md.

AN INVESTIGATION OF
PRESTRESSING CONCRETE STRUCTURES

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REQUIREMENTS FOR DEGREE OF
MASTER OF CIVIL ENGINEERING

BY

ALEXANDER PHILIP ZIONELLA

AND

BRYAN SEVERANCE PICKERTY

TROY, NEW YORK

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THE UNIVERSITY OF CHICAGO
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Thesis
23

PHYSICS DEPARTMENT
UNIVERSITY OF CHICAGO
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BY
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CHICAGO, ILLINOIS
1952

FOREWORD

The object of this thesis is three-fold; first, to present a paper in which a correlation has been made of the existing theories and design formulae pertinent to the subject of prestressed concrete; second, to apply these existing theories to the design of a particular structure and third, to compare the structure so designed to a similar structure using conventional reinforced concrete design procedures.

In the accomplishment of these objectives the authors have made no attempt to present original theories in the derivation of the design formulae.

THE
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VOLUME 31
PART 1
1901

The object of this issue is to present a paper in which a contribution has been made to the study of the human mind and its development in the light of the latest researches in the field of psychology and the allied sciences. The paper is written by a distinguished scholar and is of great interest and value to all who are concerned with the study of the human mind.

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The authors wish to express their appreciation to Associate Professor Joseph S. Kinney of the Department of Civil Engineering for his guidance in the selection of this subject and for his suggestions which aided the authors in reaching a conclusion.

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INTRODUCTION

The development of the theory of prestressing various concrete structures is not entirely new, nor is the application of the theory to actual construction unknown by any means. For certain structures such as circular pressure vessels this type of construction is widely used. For other structural members, especially straight line components, the application is much more limited at the present time. The authors feel that the advantages to be gained by using prestressing in all types of construction will eventually become more widely known, and hence the process will find ever-increasing application.

The objective of prestressing concrete construction is to eliminate concrete tensile stresses under design load conditions. This enables the designer to utilize the higher compressive stresses of modern concrete and the high tensile properties of cold drawn steel wire.

In general, the establishment of prestressing is accomplished by tensioning the steel reinforcement before the load is applied, the stretching force being transmitted to the concrete as a compressive force after the concrete has attained sufficient strength to take the stresses thus applied. In this manner, stresses of opposite sign to those occurring under load are imparted to the structure.

One of the first proposals to use prestressing was made by Jackson in 1888. The method suggested was that of strengthening the structure by tightening the reinforcement to a degree not determinate.

The development of the theory of probability has been a process of continuous growth. It is not only a theory of chance, but also a theory of the logic of science. The theory of probability is a branch of mathematics which is concerned with the study of the laws of chance. It is a theory which has been developed by many great minds, including Pascal, Fermat, Laplace, and Gauss. The theory of probability is a theory which has been applied to many different fields, including physics, chemistry, biology, and economics.

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One of the first problems in the development of probability was the problem of the fair game. This problem was first solved by Pascal and Fermat in the 17th century. The theory of probability is a theory which has been developed by many great minds, including Pascal, Fermat, Laplace, and Gauss.

The idea was carried further by Mandl who established a limit to the concrete tensile stress and to the reduction in cracking. In his work he did not, however, consider losses of the initial prestress and hence the counteraction due to stretching the reinforcement was only partially effective. Later work in this field by Dill, Hewett, Frayssinet, and others arrived at a process whereby full prestressing was utilized, all possible losses being considered and cracklessness being guaranteed due to the absence of any tensile stresses in the concrete under load.

The most recent work in prestressing aims at refinements to the early propositions. Various means have been advanced to achieve the desired degree of prestressing and the application of the process has been extended.

In general, two methods of applying the initial prestress are used; pre-stretching and post-stretching. The terms indicate whether the tensioning is carried out before or after hardening of the concrete. With pre-stretching, the concrete remains in the mold until the stretching can be transmitted to the concrete by bond. At the outset the tensile force in the steel is taken at the ends of the mold by special anchorages which are subsequently removed.

Post-stretching is applied when the concrete has hardened. In this case permanent anchorages at the ends of the structure transmit the compression to the concrete, there being no bond between the concrete and the steel.

With pre-stretching, at the release of the stretching force to the concrete, the initial prestress is immediately reduced due to the elastic deformation of the concrete and to shrinkage, which losses are increased gradually by further shrinkage and by plastic flow of the concrete. With post-stretching the losses due to elastic deformation and to the initial shrinkage are eliminated, thus the use of conventional steel is satisfactory.

In the design of prestressed structures the nature of the structure will determine which of the various systems of prestressing can be used most advantageously and which types of materials are best suited to the job. It must be decided whether full or partial prestressing will be used, based on the characteristics imparted by these two methods.

In full prestressing the stretching force is of such magnitude that no tensile stress occurs under working load and thus cracklessness of the concrete is guaranteed. This latter quality is, of course, highly desirable in pressure vessels such as pipes and tanks under pressure. Due to the fact that the concrete can be prevented from cracking the sections of concrete can be treated as a homogeneous material and the design is not, therefore, based on the "cracked section" as is the case in the conventional design. This absence of cracks and absence of tensile stresses in the concrete also increases the shear resistance of the concrete. In certain cases it allows the economical use of high strength steel. The strain of the structure at the release of compression to the concrete is greater than under working load,

[illegible]

It was decided at the meeting that the following steps should be taken:

[illegible]

which necessitates a high early compressive strength and which may cause inconvenience for transporting and handling precast products, since any additional strain has to be avoided.

In cases where absence of cracks is not necessary as in beams, for example, partial prestressing may be more advantageous. In this case a smaller stretching force is required and is applied to only part of the total reinforcement. This has an effect on the economy over full prestressing because of the reduction in fabrication costs and because of the greater ease of handling precast products. By controlling the degree of initial prestress the strain under working load (deflection in the case of beams) can be controlled so as to obtain any degree of deflection between the limits of the large deformations and heavy cracking of the non-prestressed structure and the extremely small deformation and absence of cracks in the fully prestressed structure.

In the design the losses of the initial prestress mentioned previously must be considered in order that the final compressive stress in the concrete prior to loading may be held within the limits desired. These losses are computed and allowance made for them in the particular case at hand by increasing the initial prestress in the steel.

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1. The first step in the process of identifying a problem is to define the problem. This involves identifying the symptoms of the problem and determining the scope of the problem. Once the problem has been defined, the next step is to identify the causes of the problem. This involves identifying the factors that are contributing to the problem and determining the underlying causes. Once the causes have been identified, the next step is to develop a plan of action. This involves identifying the steps that need to be taken to solve the problem and determining the resources that will be needed to implement the plan. Finally, the last step in the process is to implement the plan and monitor the results. This involves putting the plan into action and tracking the progress of the solution. Once the problem has been solved, the final step is to evaluate the results and determine if the solution was effective. This involves comparing the results of the solution to the original problem and determining if the problem has been solved. If the problem has not been solved, the process may need to be repeated.

GENERAL THEORY OF DESIGN

The application of prestressing to particular structures will now be considered. Since prestressing is most advantageously used in pressure vessels, this type of structure will be considered first.

WIRE WOUND PRESTRESSED CONCRETE PRESSURE PIPE

Conventional reinforced concrete pipes have been used in water supply lines for many years within the limitations of internal pressures and external loadings; but with the comparatively recent development of wire wound prestressed concrete pipe greater internal pressures can be carried and greater resistance to external loading afforded.

The wire wound prestressed concrete pressure pipe has advantages of economy of steel and quality of concrete to satisfy engineering designs for high pressure heads. The magnitude of internal pressure resulting from hydrostatic head and external loadings, to be resisted by the pipe will govern the design for quantity of steel wire and the amount of prestress necessary to use in the wrapping.

Refer to Figure 5;

f_s stress in the steel, p.s.i.

f'_s stress in the steel due to internal pressure, p.s.i.

f_c stress in the concrete, p.s.i.

f'_c reduction in concrete prestress, p.s.i.

A_s area of steel, square inches.

A_c area of concrete, square inches.

r internal radius of pipe, inches.

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The definition of knowledge is not a simple one. It is a complex of many factors. It is not enough to say that knowledge is true belief. It is also necessary to say that knowledge is true belief which is based on reason.

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- t wall or shell thickness, inches
 P internal pressure, p.s.i.
 E_s modulus of elasticity of steel, p.s.i.
 E_c modulus of elasticity of concrete, p.s.i.
 $n = E_s/E_c$
 d_s deformation in steel, inches per inch.
 d_c deformation in concrete, inches per inch.

1. When the shell is wound with a wire in tension there is produced:

- (a) a tensile stress in the wire, f_s .
 (b) a compressive stress in the concrete, f_c .

In this derivation the average stress in the steel and in the concrete is used. The stress in the steel must equal the stress in the concrete to maintain equilibrium, hence:

$$f_c A_c = f_s A_s$$

For 1 inch length of pipe $A_c = t$, then:

$$f_c = \frac{f_s A_s}{t}$$

2. When the prestressed pipe is subjected to internal pressure there is produced:

- (a) An increase in the steel prestress
 (b) A decrease in the concrete pre compression.

Hence, for equilibrium:

$$Pr = f'_s A_s + f'_c A_c$$

$$d_s = \frac{f'_s}{E_s} \quad \text{and} \quad d_c = \frac{f'_c}{E_c}$$

But $d_s = d_c$; therefore $\frac{f'_s}{E_s} = \frac{f'_c}{E_c}$

1. The first condition is that

2. The second condition is that

3. The third condition is that

4. The fourth condition is that

5. The fifth condition is that

6. The sixth condition is that

7. The seventh condition is that

8. The eighth condition is that

(a) The first condition is that

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10. The tenth condition is that

(a) The first condition is that

(b) The second condition is that

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The fifth condition is that

The sixth condition is that

Whence $f's = f'e \frac{A_s}{A_c}$; but $\frac{E_s}{E_c} = n$

Hence $f'_s = f'e$ which is the increase in steel stress accompanying the decrease in concrete pre compression due to internal pressure. Since the concrete is assumed to take no tensile stress the limit of $f'e$ is the magnitude of the concrete prestress f_c , therefore:

$$Pr = n f_c A_s + f_c A_c; \quad A_c = t \text{ for 1" length}$$

$$P = f_c \frac{(n A_s + t)}{r}$$

$$f_c = \frac{Pr}{n A_s + t}$$

The curves shown in Figure 1 taken from an article by Ray S. Crepps in the A.C.I. Proceedings illustrate a simple means of design for any one particular pipe which is subjected to a varying hydrostatic head. By plotting both design equations on the same set of axes a convenient and rapid solution of the equations is available. An example to illustrate the use of these curves is as follows:

Suppose a pressure pipe with a head of 250 p.s.i. is to be used. Entering the curves with 250 p.s.i. and projecting horizontally to the internal pressure curve, then vertically to the steel area curve, read directly the area of steel necessary per foot of length of pipe; in this case 0.04 square inches per foot.

$$E = \frac{1}{2} \rho \int_{-L}^L \dot{y}^2 dx$$

From (7) we find that the rate of change of the energy is given by

$$\frac{dE}{dt} = \rho \int_{-L}^L \dot{y} \ddot{y} dx$$

which is zero if the motion is periodic. This is the case if the motion is periodic in time and the length of the string is constant.

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CHAPTER IV. THE DESIGN OF PRESTRESSING

The next logical step in the use of prestressing would be in the construction of a water tank or stand pipe. The theory of design here presented was taken from an article published in the A.C.I. publication "R/C" which was written by Lt. Comd'r John L. Mason (CNC) USNR. Various other articles on the subject were studied in conjunction with the design presented in this thesis; however it is the opinion of the authors that Lt. Comd'r Mason has presented the most concise and logical development of the necessary design formulae. Only the most pertinent formulae will be presented here. For a more complete development, and for examples of uses of these formulae reference to Lt. Comd'r. Mason's article or the design herein presented is suggested.

Consider a ring of concrete with a band of steel laid snugly around its exterior surface but not stressed. By means of a turnbuckle or other mechanical device keep shortening the band until it is stressed to f_{si} , the initial steel stress. The total initial force on the steel band will be $A_s f_{si}$. This force must, of course, be in equilibrium with the forces in the concrete ring, hence:

$$A_s f_{si} = A_c f_{ci} \text{ or } f_{ci} = p f_{si} \text{ where } p = \frac{A_s}{A_c}$$

Suppose now that the pre-stressed ring is subjected to an internal hydrostatic pressure of q per unit of length and the radius of the ring is r ; then the ring tension, T in the combined concrete and steel section is;

$T = q R$. The tensile stresses due to ring tension are:

$$\text{for the concrete} = \frac{T}{(A_c + nA_s)}$$

$$\text{for the steel} = \frac{nT}{(A_c + nA_s)}$$

$$\text{where } n = \frac{E_s}{E_c}.$$

The combined final stresses are:

$$\text{for the concrete } f_c = -p f_{si} + \frac{T}{A_c (1 + np)}$$

$$\text{for the steel } f_s = f_{si} + \frac{T}{A_s (1 + np)}$$

As seen from the above equations the initial compressive stress in the concrete was reduced and the initial stress in the steel was increased. For complete insurance against cracking the usual procedure is to consider the combined stress in the concrete equal to zero. Setting the equation for final concrete stress equal to zero and solving for the initial steel stress we have:

$$f_{si} = \frac{T}{A_s (1 + np)}$$

When $f_c = 0$ the total ring tension must be taken by the steel, hence $T = A_s f_s$. Therefore for the initial steel stress we have:

$$f_{si} = \frac{f_s}{1 + np}$$

This is, however, based on the assumption that the concrete behaves as a perfectly elastic material with no shrinkage. This assumption is, of course, false and a correction to the initial stress must be taken into account due to actual shrinkage. The steel bands must be given an actual stress such that:

1. The first condition is that the function f is continuous on $[a, b]$.

$$\lim_{x \rightarrow a^+} f(x) = f(a)$$

$$\lim_{x \rightarrow b^-} f(x) = f(b)$$

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$$f_{sl} = \frac{f_s + CEs}{1 + np} \quad \text{where } C =$$

shrinkage coefficient.

(For derivation of this latter equation see "Circular Concrete Tank Without Prestressing", A.C. I. "R/C" No. 4).

Lt. Comdr. Mason indicates that, from his experience, he has found the value of C to be approximately 0.0002 for tanks above ground level. He also indicates that the value of p should be limited to approximately 0.02.

From the maximum percentage of band steel an equation for minimum wall thickness t can be developed in terms of ring tension T :

$$P = \frac{A_s}{12t} = \frac{T/f_s}{12t}$$

$$t = \frac{T}{12pf_s}$$

Hence t = the minimum wall thickness that can be used. If the steel stress is constant throughout the entire height of wall then the thickness t must remain constant. By using the relation that $A_s = \frac{T}{f_s}$ the area of steel necessary at any point in the wall can be determined.

$$\frac{d}{dt} \left(\frac{1}{2} m v^2 \right) = \frac{1}{2} m \frac{dv^2}{dt}$$

where m is the mass of the particle.

The expression for the kinetic energy of a particle is given by

$$K = \frac{1}{2} m v^2$$

where v is the velocity of the particle. The rate of change of kinetic energy is

$$\frac{dK}{dt} = \frac{1}{2} m \frac{dv^2}{dt}$$

which is the same as the rate of change of the square of the velocity.

$$\frac{dK}{dt} = m v \frac{dv}{dt}$$

From the definition of force, we have

$$F = \frac{dp}{dt}$$

where p is the momentum of the particle. The rate of change of momentum is

$$\frac{dp}{dt} = \frac{d}{dt} (m v)$$

$$= m \frac{dv}{dt}$$

where m is the mass of the particle. The rate of change of momentum is

$$\frac{dp}{dt} = m \frac{dv}{dt}$$

$$= m \frac{dv}{dt}$$

which is the same as the rate of change of the velocity.

STRAIGHT LINE MEMBERS

A more recent application of the theory of prestressing, and one that is coming into prominence, is the application of the theory to straight line members such as beams, columns, and girders.

The theory applied to these elementary components may be carried into the construction of larger structures. It has been claimed by advocates of the theory that when applied to bridges there results a structure which gives excellent resistance to the effect of concentrated wheel loads, impact and vibrations. Long span bridges of the simple beam and cantilever type can be designed with a relatively small depth-span ratio combined with small deflections (according to Schorer).

The theory herein presented is taken from an article by Herman Schorer in the Journal of the American Concrete Institute. As previously stated in the introduction there is a loss in the initial steel prestress due to the following causes:

- (1) Shrinkage of the concrete.
- (2) Plastic flow of the concrete.
- (3) Elastic deformation of the concrete.

The total change in the initial stress in the steel may be given approximately by the following empirical formula:

$$\text{Change in } f_s = 15,000 + 15 f_o \quad (a)$$

Hence the final effective steel stress is:

$$f_s = f_{so} - \text{change in } f_s$$

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(4) Gravitation of the earth.
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 (6) Gravitation of the earth.

The effective stress can be determined by means of successive approximations. For this purpose the entire original prestressed force P_0 is first assumed to be acting, whereby an approximate value of the concrete prestress is obtained. The corresponding steel stress reduction is then determined from the above equation. This approximation is then used to obtain a second approximation for the effective prestress force, and this procedure is repeated until the corrections become negligible.

A derivation of the design formulae for beams and girders follows:

Notation:

Change in f_g = total steel stress reduction.

f_g = effective concrete stress.

f_{so} = original steel stress.

P_0 = original steel prestress force.

f_{cep} = effective concrete stress at the c.g.c. due to prestress force, P .

M_p = internal moment, due to prestress force, P .

f_{c1p} = extreme concrete fiber stress due to prestress force, P .

f_{c2p} = extreme concrete fiber stress due to prestress force, P .

s_1 = S_{c1}/A .

s_2 = S_{c2}/A .

k_1 = f_{c1p}/f_{cep} , stress ratio.

k_2 = f_{c2p}/f_{cep} , stress ratio.

f_{cep} = effective concrete stress at the c.g.c. due to prestress force, P .

c_1 = extreme fiber distance from c.g.c.

c_2 = extreme fiber distance from c.g.c.

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*1. A single, continuous, linear chain of nucleotides that form a double helix.

25. The following table shows the number of people who attended the concert in each age group.

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- $k_0 = f_{c0p}/f_{c02}$, stress ratio.
 M_0 = dead load moment.
 e = distance between center gravity concrete and neutral axis.
 A = area of transformed section.
 A_s = steel area.
 f'_{c1m} = concrete fiber stress change due to external moment, M.
 f'_{c2m} = concrete fiber stress change due to external moment, M.
 f'_{cem} = concrete stress change at the c.g.c. due to external moment, M.
 f_{sm} = steel stress change due to live load moment, M.
 S_{c1} = section modulus of fiber c_1 , referred to c.g.c.
 S_{c2} = section modulus of fiber c_2 , referred to c.g.c.
 M = live load moment.

Figure 6 shows a beam section with an effective prestress force, P acting at the distance, e , from the c.g.c. The average effective concrete stress is,

$$f_{c0p} = \frac{P}{Ac} \quad (1)$$

The eccentric application causes an internal moment,

$$M_p = eP \quad (2)$$

The extreme fiber stresses are,

$$f_{c1p} = \frac{P}{Ac} + \frac{eP}{S_{c1}} \quad (3)$$

$$f_{c2p} = \frac{P}{Ac} - \frac{eP}{S_{c2}} \quad (4)$$

By designating,

$$k_1 = \frac{S_{c1}}{Ac} \quad (5)$$

$$A_1 = \frac{1}{2} \left(\frac{1}{\omega_1} + \frac{1}{\omega_2} \right) \quad (1)$$

$$A_2 = \frac{1}{2} \left(\frac{1}{\omega_1} - \frac{1}{\omega_2} \right) \quad (2)$$

$$A_3 = \frac{1}{2} \left(\frac{1}{\omega_1} + \frac{1}{\omega_2} \right) \quad (3)$$

$$A_4 = \frac{1}{2} \left(\frac{1}{\omega_1} - \frac{1}{\omega_2} \right) \quad (4)$$

$$A_5 = \frac{1}{2} \left(\frac{1}{\omega_1} + \frac{1}{\omega_2} \right) \quad (5)$$

$$A_6 = \frac{1}{2} \left(\frac{1}{\omega_1} - \frac{1}{\omega_2} \right) \quad (6)$$

$$A_7 = \frac{1}{2} \left(\frac{1}{\omega_1} + \frac{1}{\omega_2} \right) \quad (7)$$

$$A_8 = \frac{1}{2} \left(\frac{1}{\omega_1} - \frac{1}{\omega_2} \right) \quad (8)$$

$$A_9 = \frac{1}{2} \left(\frac{1}{\omega_1} + \frac{1}{\omega_2} \right) \quad (9)$$

$$A_{10} = \frac{1}{2} \left(\frac{1}{\omega_1} - \frac{1}{\omega_2} \right) \quad (10)$$

$$A_{11} = \frac{1}{2} \left(\frac{1}{\omega_1} + \frac{1}{\omega_2} \right) \quad (11)$$

$$A_{12} = \frac{1}{2} \left(\frac{1}{\omega_1} - \frac{1}{\omega_2} \right) \quad (12)$$

$$A_{13} = \frac{1}{2} \left(\frac{1}{\omega_1} + \frac{1}{\omega_2} \right) \quad (13)$$

$$A_{14} = \frac{1}{2} \left(\frac{1}{\omega_1} - \frac{1}{\omega_2} \right) \quad (14)$$

$$A_{15} = \frac{1}{2} \left(\frac{1}{\omega_1} + \frac{1}{\omega_2} \right) \quad (15)$$

$$A_{16} = \frac{1}{2} \left(\frac{1}{\omega_1} - \frac{1}{\omega_2} \right) \quad (16)$$

$$A_{17} = \frac{1}{2} \left(\frac{1}{\omega_1} + \frac{1}{\omega_2} \right) \quad (17)$$

$$A_{18} = \frac{1}{2} \left(\frac{1}{\omega_1} - \frac{1}{\omega_2} \right) \quad (18)$$

$$A_{19} = \frac{1}{2} \left(\frac{1}{\omega_1} + \frac{1}{\omega_2} \right) \quad (19)$$

$$A_{20} = \frac{1}{2} \left(\frac{1}{\omega_1} - \frac{1}{\omega_2} \right) \quad (20)$$

$$A_{21} = \frac{1}{2} \left(\frac{1}{\omega_1} + \frac{1}{\omega_2} \right) \quad (21)$$

$$A_{22} = \frac{1}{2} \left(\frac{1}{\omega_1} - \frac{1}{\omega_2} \right) \quad (22)$$

$$A_{23} = \frac{1}{2} \left(\frac{1}{\omega_1} + \frac{1}{\omega_2} \right) \quad (23)$$

$$A_{24} = \frac{1}{2} \left(\frac{1}{\omega_1} - \frac{1}{\omega_2} \right) \quad (24)$$

$$A_{25} = \frac{1}{2} \left(\frac{1}{\omega_1} + \frac{1}{\omega_2} \right) \quad (25)$$

$$A_{26} = \frac{1}{2} \left(\frac{1}{\omega_1} - \frac{1}{\omega_2} \right) \quad (26)$$

And,

$$e_2 = \frac{Ne_2}{A_0} \quad (6)$$

The fiber stresses are then determined by means of stress ratios;

$$k_1 = \frac{f_{c1p}}{f_{csp}} = \left(1 + \frac{e}{s_1} \right) \quad (7)$$

and,

$$k_2 = \frac{f_{c2p}}{f_{csp}} = \left(1 - \frac{e}{s_2} \right) \quad (8)$$

The concrete stress at the c.g.s. is,

$$f_{csp} = f_{c1p} - (f_{c1p} - f_{c2p}) \frac{(C_1 - e)}{C_1 + C_2} \quad (9)$$

which after transformation gives,

$$f_{csp} = \frac{(1 + e^2/A_0)}{I_0} \quad (10)$$

or by introducing the stress ratio,

$$k_3 = \frac{f_{csp}}{f_{csp}} = \frac{I_0}{I_0} \quad (11)$$

the effective steel stress is,

$$f_{sp} = \frac{P}{A_s} \quad (12)$$

The originally required steel stress can now be determined by substituting f_{csp} in equation (a),

$$\text{Change in } f_s = f_{csp} + 15 f_c$$

The designer is now confronted with the opposite problem; that is, the determination of the effective stress due to the release of the original prestress force, P_0 , also the simultaneous influence of dead loads.

(A)

$$\frac{1}{2} = \frac{1}{2}$$

(V)

$$\left(\frac{1}{2} = \frac{1}{2} \right)$$

(B)

$$\left(\frac{1}{2} = \frac{1}{2} \right)$$

(C)

$$\frac{1}{2} = \frac{1}{2}$$

(D)

$$\frac{1}{2} = \frac{1}{2}$$

(E)

$$\frac{1}{2} = \frac{1}{2}$$

(F)

$$\frac{1}{2} = \frac{1}{2}$$

If the dead load moment, M_0 , is smaller than the counteracting prestress moment, equation (2), the release of the prestress force will cause the beam to lift off the supporting forms. In this case the dead load acts simultaneously with the prestress load and the steel stress loss is determined by the combined concrete stress at the c.g.c. The concrete stress, f_{c0M_0} , due to dead load moment is,

$$f_{c0M_0} = \frac{M_0}{S_c} \quad (13)$$

The effective stresses are preferably determined by the method of successive approximations. For this purpose the known value of P_0 , instead of P is at first substituted in equation (1), which results in an approximate stress f_{ceP} as derived from equation (11). This stress is combined with the given dead load stress, equation (13). The resulting stress, substituted in change of prestress equation, gives an approximation for the total steel stress reduction; therefore a correction for the effective prestress force.

The effective fiber stresses are now obtainable from equations (7) and (8) by means of the final value of P , as substituted in equation (1). In the case of top and bottom reinforcing the corrections should be made simultaneously, although the two prestress forces can be treated independently as a first approximation.

The stress changes due to live loads must now be considered. The stress changes due to live loads are based on the transformed section, assuming that volume changes can be considered as completed. The transformed area, A , is based on the entire concrete section,

$$A = A_c + nA_s \quad (14)$$

It was found that the...

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With reference to Figure 6 the location of the neutral axis is given by the relation,

$$a = \frac{c n A_s}{A} \quad (15)$$

$$I = I_c + a^2 A_c + (e - a)^2 n A_s \quad (16)$$

or,

$$I = I_c + a e a c \quad (17)$$

With $c'1 = c_1 - a$ the section modulus for the bottom fiber becomes,

$$S'_{c1} = \frac{I}{c'1} \quad (18)$$

For the top fiber, with $c'2 = c_2 + a$,

$$S'_{c2} = \frac{I}{c'2} \quad (19)$$

For the c.g.s. with $e' = e - a$,

$$S'_e = \frac{I}{e'} \quad (20)$$

The concrete stress changes produced by the live load moment, M , then are,

$$f'_{c1m} = \frac{M}{S'_{c1}} \quad (21)$$

$$f'_{c2m} = \frac{M}{S'_{c2}}$$

$$f'_{cem} = \frac{M}{S'_e}$$

and the steel stress change, from equation $f_s = n f_c$ becomes,

$$f_{sm} = n f'_{cem}.$$

(11)

$$\frac{1}{\Delta} = \frac{1}{\Delta}$$

(12)

$$I = I_0 + \frac{1}{2} I_0 \frac{v^2}{c^2} + \frac{1}{8} I_0 \frac{v^4}{c^4} + \dots$$

(13)

$$I = I_0 + \frac{1}{2} I_0 \frac{v^2}{c^2} + \frac{1}{8} I_0 \frac{v^4}{c^4} + \dots$$

(14)

$$\frac{1}{I} = \frac{1}{I_0} + \frac{1}{2} \frac{v^2}{c^2} \frac{1}{I_0} + \dots$$

(15)

$$\frac{1}{I} = \frac{1}{I_0} + \frac{1}{2} \frac{v^2}{c^2} \frac{1}{I_0} + \dots$$

(16)

$$\frac{1}{I} = \frac{1}{I_0} + \frac{1}{2} \frac{v^2}{c^2} \frac{1}{I_0} + \dots$$

(17)

$$\frac{1}{I} = \frac{1}{I_0} + \frac{1}{2} \frac{v^2}{c^2} \frac{1}{I_0} + \dots$$

$$\frac{1}{I} = \frac{1}{I_0} + \frac{1}{2} \frac{v^2}{c^2} \frac{1}{I_0} + \dots$$

$$\frac{1}{I} = \frac{1}{I_0} + \frac{1}{2} \frac{v^2}{c^2} \frac{1}{I_0} + \dots$$

$$I = I_0 + \frac{1}{2} I_0 \frac{v^2}{c^2} + \frac{1}{8} I_0 \frac{v^4}{c^4} + \dots$$

COMPARATIVE DESIGN

The purpose of this comparative design is to arrive at a measure of the relative economy of materials required for a structure designed first, by conventional reinforced concrete design procedure and second, by using the theories of prestressed structures. For the purpose of an overall economic study such a comparison of designs should, of course, take into account the relative costs of fabrication and the labor charges. A determination of these latter factors is, however, beyond the scope of this study; hence the comparison will be based solely upon the quantity of materials required in each case.

For the purpose as outlined above the structure to be compared will be a 500,000 gallon circular water storage tank. The comparison will be extended in the case of the tank cover to include a flat slab cover in one case and a segmental dome cover in the other. These types were chosen as being best suited to the individual tank designs.

Notation

- A_c = area of concrete section, square inches.
- A_s = area of steel in tension, square inches.
- b = width of rectangular beam, inches.
- ϵ = shrinkage coefficient of concrete.
- d = effective depth, inches.
- D = diameter of tank, feet.
- E_c = modulus of elasticity of concrete in compression, p.s.i.
- E_s = modulus of elasticity of steel, p.s.i.

REMARKS

The first of the two main objects of the present report is to describe the results of the investigation of the physical properties of the various specimens of the material under consideration. The second object is to compare the results of the present investigation with the results of previous investigations of the same material. The third object is to discuss the results of the present investigation in relation to the theory of the physical properties of the material under consideration. The fourth object is to discuss the results of the present investigation in relation to the theory of the physical properties of the material under consideration.

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RESULTS

1. The first of the two main objects of the present report is to describe the results of the investigation of the physical properties of the various specimens of the material under consideration.
2. The second object is to compare the results of the present investigation with the results of previous investigations of the same material.
3. The third object is to discuss the results of the present investigation in relation to the theory of the physical properties of the material under consideration.
4. The fourth object is to discuss the results of the present investigation in relation to the theory of the physical properties of the material under consideration.

f_c = compressive unit stress in extreme fiber of concrete, p.s.i.
 f'_c = ultimate compressive strength of concrete, p.s.i.- at age of 28 days.
 f_s = tensile unit stress in longitudinal reinforcement, p.s.i.
 f_{s1} = initial stress in steel, p.s.i.
 f_{c1} = initial stress in concrete, p.s.i.
 H = height of wall, feet.
 H_1 = hoop stress in dome at point 1, kips per foot.
 I = moment of inertia of horizontal cross section of tank, inch units.
 I_s = moment of inertia of reinforcement about neutral axis, inch units.
 j = ratio of distance between centroid of compression and center of gravity of tensile reinforcement to depth "d",
 k = ratio of distance between the compressive face of the beam and the neutral axis to the depth "d".
 M = moment due to dead and live loads, foot pounds.
 n = ratio of modulus of elasticity of steel to that of concrete.
 p = ratio of area of tensile reinforcement to the effective area of concrete in beams and slabs.
 r = radius of sphere, feet.
 R = radius of tank, feet.
 S_0 = compression in edge member of lantern opening of dome, kips.
 S_1 = ring tension in edge member of dome, kips.
 t = thickness of beam or wall, inches.
 T = hoop stress, pounds.
 T_1 = meridional stress in dome, kips per foot.

1. The first of these is the fact that the system is not a simple one, but a complex one, involving many factors.
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19. The nineteenth is the fact that the system is not a simple one, but a complex one, involving many factors.
20. The twentieth is the fact that the system is not a simple one, but a complex one, involving many factors.

v = unit shear in concrete, p.s.i.

V = total shear, pounds.

w = equivalent water pressure, pounds per cubic foot.

w_1 = combined dead and live load on dome, kips per square foot.

W_1 = weight of dome, kips.

ϕ_0 = angle subtended by axis of dome and edge of lantern opening, degrees.

ϕ_1 = angle subtended by axis of dome and support edge, degrees.

Δ = deflection, inches.

1. The first part of the report is a general statement of the purpose and scope of the study. It is followed by a brief review of the literature on the subject.

2. The second part of the report is a description of the methods used in the study. This includes a description of the subjects, the materials, and the procedures.

3. The third part of the report is a presentation of the results. This includes a description of the data and a discussion of the findings.

4. The fourth part of the report is a conclusion. This includes a summary of the findings and a discussion of their implications.

5. The fifth part of the report is a list of references. This includes a list of the books, articles, and other sources used in the study.

DESIGN DATA

REINFORCED CONCRETE TANK:

- (1) Circular tank resting on ground surface.
- (2) Allowable soil bearing - 3000 psf.
- (3) Allowable stress in circumferential wall steel - 18,000 psi.
(This low allowable stress is chosen to prevent excessive elongation of the steel in case the concrete cracks).
- (4) Allowable stress in all other steel - 18,000 psi.
- (5) f'_c - 3000 psi.
- (6) E_s - 30,000,000 psi.
- (7) Ultimate concrete tensile strength - 250 psi.
- (8) Required tank height (effective) - 20 feet.
- (9) Assume base is fixed.
- (10) Specifications: A.C.I. 318-41 and Br. I.C.C. Specs. - 318.
- (11) Unit weight of concrete - 150 pounds per cubic foot.
- (12) Live Load on cover - 25 psf.
- (13) Wind pressure - $5/8 \times 60$ psf of vertical projection.

PRESTRESSED CONCRETE TANK:

- (1) Same diameter and effective height as R.C. tank. Same soil conditions.
- (2) Allowable stress in prestressing steel - 22,500 psi. prior to prestressing.
- (3) Allowable stress in reinforcing steel - 18,000 psi.
- (4) f'_c - 3000 psi.

PRINTED MATTER CASE

- (5) E_c - 30,000,000 psi.
- (6) Ultimate concrete tensile strength - 350 psi.
- (7) Assume base is fixed.
- (8) Unit weight of concrete - 150 pounds per cubic foot.
- (9) Specifications: A.C.I. 318-41
- (10) Live Load on cover - 30 psf.
- (11) Sellar load on dome - 15 kips.
- (12) Wind stress - $2/8 \times 50$ psf of vertical projection

REINFORCED CONCRETE

MINIMUMS:

$$\text{Vol.} = 500,000 \text{ gal.} = 64,000 \text{ cu. ft.}$$

$$\text{Diameter} = 64' - 0" \quad \text{Height} = 20'$$

THICKNESS OF WALL:

$$\text{Pressure at bottom of wall} = 62.5 \times 20 = 1250 \text{ p.s.f.}$$

$$\text{Pressure 1 ft. up from bottom} = 62.5 \times 19 = 1187 \text{ p.s.f.}$$

$$\text{Ave. for 1 ft. section} = \frac{1}{2} (1250 + 1187) = 1218 \text{ psf}$$

$$T \text{ for 1 ft. section} = 1218 \times \frac{64}{2} = 39,000^{\#}$$

To allow for shrinkage in the concrete assume coefficient of shrinkage $C = 0.0004$

$$\begin{aligned} \delta &= \frac{f_s}{n s e C + f_s - f_{en}} = \\ &= \frac{250}{10(8,000,000) (.0004) + 12000 - 250 \times 10} \\ &= 0.0116 \end{aligned}$$

$$t = \frac{T}{12 f_s} = \frac{39,000}{12 \times .0116 \times 12,000} = 23.4"$$

$$\text{Use } t = 24"$$

$$\text{Let thickness at top} = 8"$$

PROBLEM 1

Let $f(x) = x^2 + 1$ and $g(x) = x^2 - 1$.

Find the domain of the function $h(x) = \frac{f(x)}{g(x)}$.

$$f(x) = x^2 + 1 \quad g(x) = x^2 - 1$$

$$h(x) = \frac{f(x)}{g(x)} = \frac{x^2 + 1}{x^2 - 1}$$

The domain of $h(x)$ is the set of all real numbers x such that $g(x) \neq 0$.

Therefore, the domain of $h(x)$ is $\{x \in \mathbb{R} \mid x^2 - 1 \neq 0\}$.

$$x^2 - 1 \neq 0 \iff x^2 \neq 1 \iff x \neq \pm 1$$

$$\text{Therefore, the domain of } h(x) \text{ is } \{x \in \mathbb{R} \mid x \neq \pm 1\}.$$

$$\text{We can also write the domain as } \mathbb{R} \setminus \{-1, 1\}.$$

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$$\text{The domain of } h(x) \text{ is } \mathbb{R} \setminus \{-1, 1\}.$$

AREA OF STEEL:

Assume concrete cracks and steel takes all of load

$$A_s = \frac{59000}{12000} = 4.92 \text{ sq. in. per ft. of ht.}$$

Per inch of height:

$$A_s = \frac{4.92}{12} = 0.41 \text{ sq. in. per inch.}$$

$$A_s \text{ each side} = \frac{0.41}{2} = 0.205$$

Try $7/8" \phi$

$$\text{Spacing} = \frac{0.6013}{0.205} = 2.93 \text{ USE } 3"$$

Change Steel spacing at 5 ft. intervals.

At 5 ft. above base:

$$T = 62.5 \times 15 \times \frac{54}{2} = 25000 \text{ \# per ft. of ht.}$$

$$A_s = \frac{25000}{12000} = 2.08 \text{ sq. in. per ft.}$$

$$A_s = \frac{2.08}{12} = 0.173 \text{ sq. in. per inch.}$$

$$\text{For two rows } A_s = 0.087 \text{ in. per in.}$$

For $7/8" \phi$

$$\text{Spacing} = \frac{0.6013}{0.087} = 6.91 \text{ USE } 7 \text{ inches}$$

At 10 ft. above base

$$T = 62.5 \times 10 \times \frac{54}{2} = 16667 \text{ \# per ft. of ht.}$$

$$A_s = \frac{16667}{12000} = 1.39 \text{ sq. in. per ft.}$$

$$A_s = 0.116 \text{ sq. in. per in.}$$

1944-1945

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for two rows $\Delta s = 0.0495$ in. per in.

For $7/8 \text{ } \phi$

$$\text{Spacing} = \frac{0.6013}{0.0695} = 8.65 \text{ USE } 9\frac{1}{2} \text{ in.}$$

At 15 ft. above base:

$$T = 62.5 \times 5 \times \frac{64}{2} = 10,000 \text{ } \phi$$

$$\Delta_s = \frac{10,000}{12,000} = 0.833 \text{ in. per ft.}$$

$$\Delta s = 0.0695 \text{ inches per in.}$$

For two rows $\Delta s = 0.0343$ in. per in.

For $5/8 \text{ } \phi$

$$\text{Spacing} = \frac{0.31}{0.0343} = 8.9 \text{ in. USE } 9\frac{1}{2} \text{ in.}$$

For the year 1912 the total was

\$ 1,000,000

For the year 1913 the total was

\$ 1,000,000

For the year 1914 the total was

\$ 1,000,000

For the year 1915 the total was

\$ 1,000,000

\$ 1,000,000

For the year 1916 the total was

\$ 1,000,000

For the year 1917 the total was

\$ 1,000,000

For the year 1918 the total was

\$ 1,000,000

For the year 1919 the total was

\$ 1,000,000

For the year 1920 the total was

MOMENT IN WALL DUE TO FILLING DAM:

$$\begin{aligned}\text{Max. deflection} &= n f_o \frac{p}{2E_s} = \frac{2500 \times 64}{2 \times 30,000,000} \\ &= 0.00267 \text{ ft.} \\ &= 0.032 \text{ in.}\end{aligned}$$

Allowing 2" Cover for steel.

$$A_G = 24 \times 12 = 288 \text{ sq. in.}$$

$$\text{Take } A_s = 0.01 A_G = 0.01 \times 288 = 2.88 \text{ sq. in.}$$

$$t = 24"$$

$$p = 62.5 \times 20 = 1250 \text{ p.s.f.} = 104 \text{ p/in. of ft.}$$

$$I_s = 2.88 \times 10 \times 10 = 288 \text{ inch units}$$

$$I_G = \frac{1}{12} \times 12 \times 24^3 + 9 \times 288 = 13,600 + 2590$$

$$I_G = 16,190 \text{ inch units}$$

Let h_1 = height above base to which cantilever action extends.

$$\text{Deflection} = \frac{1}{80} \frac{(p^1 h_1^4)}{EI}$$

$$\text{or } h_1 = 124.5 (5.05)^{\frac{1}{4}} = 186.5 \text{ in.}$$

$$h_1 = 15.55 \text{ ft.}$$

MOMENT AT BASE:

$$M_1 = \frac{104 \times (186.5)^2}{8} = 451,000 \text{ in p.}$$

$$\text{Taking } d = 20"$$

$$j = 0.857$$

$$k = 0.439$$

$$A_s f_s = P = \frac{451,000}{0.857 \times 20} = 26,400 \text{ p}$$

Assume $f_c = 18,000$ p.s.c.

$$A_g = \frac{25400}{18000} = 1.465 \text{ sq. inches for 1 ft. section.}$$

$$A_s = 0.122 \text{ sq. inches / inch}$$

For $7/8 \phi$

$$\text{Spacing} = \frac{0.6013}{0.122} = 4.93 \text{ USE } 4 \text{ in.}$$

POINT OF CONTRAFLEXURE:

$$\text{Point of C.F.} = 0.37 h_1$$

$$= 0.37 \times 15.53 = 5.75 \text{ ft. up from base.}$$

Extend rods to 7 ft. above base

COMPRESSIVE STRESS IN CONCRETE:

$$f_c = \frac{2M}{k_j b d^2} = \frac{2 \times 451,000}{0.429 \times 0.857 \times 12 \times 400}$$

$$= 511 \text{ p.s.c.} \approx \text{Bott}$$

MAXIMUM MOMENT ABOVE POINT OF CF.

$$M = 1/3 M_1$$

$$A_s = 1/3 A_{s1} = \frac{0.122}{3} = 0.041 \text{ sq. inches/inch}$$

for $1/2 \text{ in } \phi$

$$\text{Spacing} = \frac{0.20}{0.041} = 4.88 \text{ USE } 4 \frac{1}{2} \text{ in}$$

EXTEND TO $16 \frac{1}{2}$ Ft. ABOVE BASE.

Extend every fourth rod inside & outside to top for support of circumferential steel and to use as temperature steel.

BEAR AT TOP OF BASE:

$$V = \frac{1250}{2} \times 15.53 = 9700 \text{ #/Ft. of width.}$$

$$v = \frac{V}{b_j d} = \frac{9700}{12 \times 0.857 \times 20} = 47 \text{ psi.}$$

Allowable = 60 psi

1. $\frac{1}{2} \times \frac{1}{2} = \frac{1}{4}$

2. $\frac{1}{2} \times \frac{1}{2} = \frac{1}{4}$

3. $\frac{1}{2} \times \frac{1}{2} = \frac{1}{4}$

4. $\frac{1}{2} \times \frac{1}{2} = \frac{1}{4}$

5. $\frac{1}{2} \times \frac{1}{2} = \frac{1}{4}$

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8. $\frac{1}{2} \times \frac{1}{2} = \frac{1}{4}$

9. $\frac{1}{2} \times \frac{1}{2} = \frac{1}{4}$

10. $\frac{1}{2} \times \frac{1}{2} = \frac{1}{4}$

11. $\frac{1}{2} \times \frac{1}{2} = \frac{1}{4}$

12. $\frac{1}{2} \times \frac{1}{2} = \frac{1}{4}$

13. $\frac{1}{2} \times \frac{1}{2} = \frac{1}{4}$

14. $\frac{1}{2} \times \frac{1}{2} = \frac{1}{4}$

15. $\frac{1}{2} \times \frac{1}{2} = \frac{1}{4}$

16. $\frac{1}{2} \times \frac{1}{2} = \frac{1}{4}$

17. $\frac{1}{2} \times \frac{1}{2} = \frac{1}{4}$

18. $\frac{1}{2} \times \frac{1}{2} = \frac{1}{4}$

19. $\frac{1}{2} \times \frac{1}{2} = \frac{1}{4}$

20. $\frac{1}{2} \times \frac{1}{2} = \frac{1}{4}$

21. $\frac{1}{2} \times \frac{1}{2} = \frac{1}{4}$

22. $\frac{1}{2} \times \frac{1}{2} = \frac{1}{4}$

23. $\frac{1}{2} \times \frac{1}{2} = \frac{1}{4}$

DESIGN FOR JOINT:

Assuming simple supports at wall centerline and center of supporting column. USE Span of 32' - 0"

Snow and live load = 25 psf

Wt. of concrete = 150 #/cu. ft.

Total load = 175 #/cu. ft.

$$\text{Reaction @ Wall} = 175 \times \frac{32}{2} \times \frac{2}{3} \times \frac{32}{32} = 1065\#$$

$$\text{Reaction @ Column} = 175 \times \frac{32}{2} \times \frac{1}{3} \times \frac{32}{32} = 934\#$$

Maximum moment @ 0.577 L from Small end.

Max. Moment = 0.1283 WL

$$= 0.1283 \times 175 \times \frac{32}{2} \times 32 = 11,500 \text{ Ft. #}$$

WIDTH OF SECTION AT MAX. MOMENT

Section of max. moment = 0.577 x 32 = 18.5' from small end.

$$\text{Width} = \frac{18.5}{32} \times 12 = 6.94"$$

$$f_c = \frac{2M}{kjb d^2}$$

$$d^2 = \frac{2M}{f_c kjb} = \frac{2 \times 11,500 \times 12}{1350 \times 0.429 \times 0.857 \times 6.94}$$

$$d = 9.95" \text{ USED } = 9"$$

USE 3" Cover Total D = 12"

$$A_s = \frac{M}{f_s j d} = \frac{11,500 \times 12}{18,000 \times 0.857 \times 9} = 0.995 \text{ sq. in.}$$

$$A_s / \text{inch} = \frac{0.995}{6.94} = 0.1432 \text{ sq. in.}$$

USE 7/8" ϕ

$$\text{Spacing} = \frac{0.6015}{0.1432} = 4.2 \text{ in. USE } 4.25"$$

Check rebar spacing at wall

$$\frac{18.5}{32} = \frac{4.25}{x} \quad \text{Let } x = \text{spacing @ wall}$$

$$x = \frac{4.25 \times 32}{18.5} = 7.38 \quad \text{Use } 7$$

SHEAR AT WALL:

$$R = 1865 \text{ \#}$$

$$v = \frac{V}{b_j d} = \frac{1865}{12 \times 0.857 \times 9} = 201 \text{ psi.}$$

WALL BEARING CHECK:

$$\text{Bearing} = \frac{1865}{8 \times 12} = 19.4 \text{ psi.}$$

Shear and bearing at column end of cover will be checked after design of column.

DESIGN OF BASE:

Assume:

$$\text{footing width} = 6' - 0"$$

$$\text{Allowable soil bearing pressure} = 3000 \text{ psf.}$$

$$\text{Depth of footing} = 20 \text{ inches.}$$

LOADS:

$$\text{Load on top of wall per foot of wall} = 1865 \text{ \#}$$

$$\text{Wt. of wall per foot} = 4000 \text{ \#}$$

$$\text{Wt. of base} = 1490 \text{ \#}$$

$$\text{Total} \quad \quad \quad \underline{7355 \text{ \#/ft. of wall}}$$

$$\text{Soil bearing due to dead load} = \frac{7355}{6} = 1226 \text{ psf}$$

Given: $\alpha = 0.05$

$$\text{Test Statistic } Z = \frac{\bar{X} - \mu_0}{\sigma / \sqrt{n}} = \frac{1.01 - 1.0}{0.01} = 10$$

$$\text{Critical Value } Z_{\alpha/2} = \frac{Z_{0.025}}{0.01} = 1.96$$

$$\text{Decision: } Z > Z_{\alpha/2}$$

Reject H_0

$$\text{p-value} = \frac{1}{2} (1 - \Phi(10)) = \frac{1}{2} (0) = 0$$

$$\text{Conclusion: } \text{p-value} < \alpha \Rightarrow \text{Reject } H_0$$

$$\text{Confidence Interval} = \bar{X} \pm Z_{\alpha/2} \frac{\sigma}{\sqrt{n}} = 1.01 \pm 1.96 \times 0.01$$

95% Confidence Interval for μ is $(0.9904, 1.0296)$

Interpretation:

We are 95% confident that the true mean is between 0.9904 and 1.0296.

Since the confidence interval does not contain the hypothesized mean $\mu_0 = 1.0$, we reject H_0 .

Conclusion: $\mu \neq 1.0$

$$\mu \neq 1.0$$

Alternative hypothesis: $\mu \neq 1.0$

Null hypothesis: $\mu = 1.0$

QED

Since we reject H_0 , we conclude that $\mu \neq 1.0$.

95% Confidence Interval for μ is $(0.9904, 1.0296)$.

Interpretation:

We are 95% confident that the true mean is between 0.9904 and 1.0296.

$$\text{95\% Confidence Interval for } \mu \text{ is } \bar{X} \pm Z_{\alpha/2} \frac{\sigma}{\sqrt{n}} = 1.01 \pm 1.96 \times 0.01$$

Total load due to water on base:

$$\pi \times 32 \times 32 \times 1250 = 4,020,000 \text{ \#}$$

Area of base:

$$\pi \times 36 \times 36 = 4070 \text{ sq. ft.}$$

Soil bearing due to water:

$$\frac{4,020,000}{4070} = 987 \text{ psf}$$

Total soil bearing pressure at foot of wall:

$$987 + 1226 = 2213 \text{ psf.}$$

3000 psf allowed.

Considering the part of the foundation that extends beyond the wall to act as a cantilever and taking moments about the face of the wall we have -

$$M = 12 \times (2213) \times 6 = 53,100 \text{ in. lbs.}$$

$$A_s = \frac{M}{f_{sy}d} = \frac{53,100}{18000 \times 0.857 \times 17}$$

$$A_s = 0.202 \text{ sq. in. per foot}$$

Using $\frac{1}{4}$ " ϕ Area = 0.20

$$\text{Spacing} = \frac{0.20}{0.202} = .99 \text{ ft.}$$

USE 12" Spacing for $1/2$ " ϕ

SHEAR CHECK AT FACE OF WALL:

$$v = \frac{4420}{12 \times 0.857 \times 17} = 25.3 \text{ psi}$$

Allowable = 60 psi.

CHECK FOR WIND LOAD:

Assume 50 #/ sq. ft.

total cost due to entry on same;

It is also to be noted that

the cost of entry is

It is also to be noted that

the cost of entry is

$\frac{1,000,000}{100} = 10,000$

Total cost of entry is

10,000

10,000

Consequently the cost of the transaction is

the cost of entry is

10,000

$10,000 \times 1.1 = 11,000$

$\frac{11,000}{1.1} = 10,000$

$10,000 \times 1.1 = 11,000$

11,000

$\frac{11,000}{1.1} = 10,000$

10,000

10,000

$\frac{10,000}{1.1} = 9,090.91$

9,090.91

9,090.91

9,090.91

$$\text{Load} = 5/8 \times 50 \times 64 = 2000 \text{ \# /ft. of Rt.}$$

$$\text{Total} = 20 \times 2000 = 40,000 \text{ \#}$$

$$\text{Mom.} = 40000 \times 10 = 4000,000 \text{ Ft. \#}$$

MOMENT OF INERTIA OF CROSS SECTION:

$$I = \frac{\pi}{4} \left(\frac{4}{33.33}^4 - \frac{4}{32}^4 \right) = 145,000 \frac{\text{in}^4}{\text{ft.}}$$

STRESS DUE TO LIVE LOAD:

$$f = \frac{MC}{I} = \frac{400,000 \times 33.33}{145,000} = 91.5 \text{ \# /ft. of wall}$$

This additional loading would not change dimensions of footing.

COLUMN DESIGN:

$$\text{Roof slab reaction on column} = 188,000 \text{ \#}$$

Estimated weights:

$$\text{Capital} = 4530 \text{ \#}$$

$$\text{Drop panel} = 3200 \text{ \#}$$

$$\text{Column} = 5560 \text{ \#}$$

$$\text{Total} = 13,290$$

$$\begin{aligned} \text{Therefore load on base slab} &= 13,290 + 188,000 \\ &= 201,290 \end{aligned}$$

For 20" column;

$$\text{Load on concrete} = 212,000 \text{ \# from ACI Design Hand book}$$

$$\text{Theoretical load on the steel} = 0$$

Use 3/8" hoops on 12" Centers and

$$\text{Vertical Steel} = 6 - 5/8 \text{ \#}$$

$$1000 \times 10^3 \times 10^3 = 1000 \times 10^6 = 10^9$$

$$1000 \times 10^3 = 10^6$$

$$1000 \times 10^3 = 10^6$$

1000 x 10^3 = 10^6

$$\frac{1}{1000} \times 10^6 = \frac{1}{1000} \times 10^6 = 10^3$$

1000 x 10^3 = 10^6

$$1000 \times 10^3 = 10^6$$

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1000 x 10^3 = 10^6

1000 x 10^3 = 10^6

Size of column and steel, though not required to carry load were chosen to lend rigidity which is the important factor.

Soil bearing at base of column.

Use 10' -0" Dia. base slab.

Area = 78.6 sq. ft.

Wt. of Base $78.6 \times 1 \times 150 = 11,800 \text{ \#}$

Total wt. on soil = $11,800 + 201,290 = 213,090$

Stress = $\frac{213,090}{78.6} = 2,710 \text{ p.s.f.}$

CHECK AT POINT AND BEARING IN TERMS OF COLUMN CAPITAL:

Assume bearing at 2 ft. from center of column.

Bearing area = 11.25 sq. in.

Reaction on column = 934 \#

Bearing pressure = $\frac{934}{11.25} = 83.4 \text{ psi}$

SHEAR CHECK:

At a distance $d = t - 1\frac{1}{2}$ from edge of column. Capital $b = 1.45'$

$d = 10.5'$

$v = \frac{V}{bjd} = \frac{934}{1.45 \times 0.657 \times 10.5} = 71.5 \text{ psi.}$

Allowable = 90 psi.

DESIGN OF FLOOR SLAB:

Except for the cantilever section of the base no moment occurs in the base slab. Assuming uniform settlement of the soil. Therefore the slab will not be designed to resist moment. However, to assume the settlement over the entire area to be uniform might lead to

also be subject to the same conditions as the other two.

It is also subject to the same conditions as the other two.

The first of these conditions is that the

second of these conditions is that the

third of these conditions is that the

fourth of these conditions is that the

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It is also subject to the same conditions as the other two.

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DRAWING OF BASE SLAB - Continued :

dangerous cracking of the floor of the tank. To prevent this a relatively thin slab will be laid. The flexibility of the thin slab will be such that it can conform to any slight differential settlement without producing cracks.

(See drawing for dimensions and steel in base slab)

Also all I can say is that I am sorry to hear that you are not well. I hope you will be able to get back to work soon. I am sure you will be able to do so. I am sure you will be able to do so. I am sure you will be able to do so.

1. The Commission has received information from the
2. Ministry of Health, that the following persons have
3. been identified as having been in contact with the
4. patient during the period of the outbreak.

PRESTRESSED DESIGN

WALL THICKNESS & STEEL AREA:

Pressure at bottom of wall

$$P = WER = 62.5 \times 20 \times 32 = 40,000 \text{ lb./ft.}$$

Assuming the concrete to take zero stress when Tank is full:

$$A_s = \frac{P}{f_y} = \frac{40,000}{22,500} = 1.78 \text{ sq. in./ft.}$$

(See Fig. 2 for steel spacings)

Adding 1 foot at bottom of tank for construction joint and using

4 bands @ 5" spacing, total number of bands used will be 41 3/4" ϕ .

INITIAL STRESS:

$$f_{si} = \frac{f_c + \text{CRS}}{1 + n\mu} = \frac{22,500 + \frac{0.0002 \times 30 \times 10,000,000}{1.14}}{1 + n\mu}$$

= 25,000 psi allowable initial steel prestress.

WALL THICKNESS:

Current practice limits percentage of steel in bands to 2% of concrete area; hence

$$t = \frac{1.78}{0.02 \times 12} = 7.54" \text{ U.S. 8"}$$

$$\text{Max. percentage of steel} = \frac{1.76}{8 \times 12} = .01833$$

CHOICE OF INITIAL CONCRETE STRESS:

Since $f_{ci} = - p f_{ci}$.

$$f_{ci} = -.01833 \times 25,000 = 458 \text{ psi Allowable} = 1350 \text{ psi}$$

Minimum steel percentage at top of wall

$$\frac{0.33}{8 \times 12} = 0.00344 \text{ Therefore}$$

$$f_{ci} (\text{Min}) = -0.00344 \times 25,000 = 86 \text{ psi.}$$

PROBLEM 1

SOLUTION

Let x be the number of units of product A.

$$x = 1000 - 2000 + 1000 = 0$$

Let y be the number of units of product B.

$$y = 1000 - 2000 + 1000 = 0$$

Let z be the number of units of product C.

Let w be the number of units of product D.

Let v be the number of units of product E.

PROBLEM 2

$$x = 1000 - 2000 + 1000 = 0$$

Let y be the number of units of product B.

PROBLEM 3

Let x be the number of units of product A.

Let y be the number of units of product B.

$$x = 1000 - 2000 + 1000 = 0$$

$$y = 1000 - 2000 + 1000 = 0$$

PROBLEM 4

Let x be the number of units of product A.

Let y be the number of units of product B.

Let z be the number of units of product C.

$$x = 1000 - 2000 + 1000 = 0$$

Let y be the number of units of product B.

TENSION BAND LENGTH:

Radius to center line of bands

$$\text{Radius} = 32 + \frac{2 + .375}{12} = 32.692 \text{ ft.}$$

$$\text{Circumference} = 2\pi \times 32.692 = 206.0 \text{ ft.}$$

USE 4 bars per band length 51.5' per bar.

$$\text{Total number of bars} = 4 \times 41 = 164 = 51.5' \text{ and } 164 \text{ turn buckles}$$

CHECK FOR TENSILE STRESS W/IN TO PRESTRESSING BANDS:

When the bottom bands are prestressed to a ring tension of

$$1.76 \times 25,000 = 44,000 \text{ \#/ft.}$$

they will exert a radial pressure on the wall equal to their ring tension divided by their radius,

$$\frac{44,000}{32.69} = 1347 \text{ \#/ft.}$$

This corresponds to a water pressure of

$$\frac{1347}{20} = 67.3 \text{ \#/ft.}$$

Using moment coefficient tables given in "MODERN DEVELOPMENTS IN REINFORCED CONCRETE".

Entering table with:

$$\frac{h^2}{Dt} = \frac{20 \times 20}{64 \times 0.667} = 9.36$$

$$\text{And } Wt^3 = 67.3 \times \frac{20^3}{20} = 538,000 \text{ ft. \#/ft.}$$

Point	0.0H	0.1H	0.2H	0.3H	0.4H	0.5H	0.6H	0.7H	0.8H	0.9H	1.0H
Coeff.											
from	0.0	0.0	0.0	-.00002	-.00001	.00004	.00014	.00029	.00047	.00049	0
Table											
VIII											
Moment	0	0	0	-107.6	-55.8	124.5	154	1560	2530	2640	0

Section 10 of Chapter 10 of the

for 202,000 = 255,414.25 = million

ART 8.342 - 242.00.01 - 2000-00000000

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1974-1975 000,000 000,000 000,000 000,000

They will want to know if we are showing them a good life.

[illegible]

$$x^{\frac{1}{2}} \frac{dx}{dt} = \frac{1}{2} \frac{dx^{\frac{1}{2}}}{dt}$$

20. *Conclusions* and *Recommendations*

• 10/10 6.70 • 10.1

$$\lim_{n \rightarrow \infty} \frac{1}{n^2} = 0, \quad \lim_{n \rightarrow \infty} \frac{1}{n} = 0, \quad \lim_{n \rightarrow \infty} \frac{1}{\sqrt{n}} = 0$$

[illegible]

0 6485 0099 7401 501 8.001 8.001- 8.001- 0 - 0 - 0

See Moment curve fig. 3. (appended)

From Moment diagram max. moment is,

2650 ft. # @ 17.5' from top.

Vertical steel necessary to resist moment

$$A_s = \frac{M}{f_s j d} = \frac{2650 \times 12}{18000 \times 0.857 \times 6} = .344 \text{ sq. in./ft.}$$

USE 1/2" # @ 7" spacing.

Compressive concrete stress due to moment,

$$f_c = \frac{2650 \times 12 \times 4}{\frac{1}{12} \times 12 \times \frac{4}{3}} = 244 \text{ p.s.i.} \quad \text{Allowable} - 250 \text{ p.s.i.}$$

DESIGN OF WALL FOOTING

Load on top of wall per ft.	=	1890 [#]
Wt. of wall per ft.	=	2100 [#]
Assumed depth of footing 12" wt.	=	600 [#]
Total		<u>4290[#]</u>

Assume width of footing = 4' -0"

Soil bearing = $\frac{4290}{4} = 1071 \text{ p.s.f.}$

Total load due to water = 4,020,000[#]

Area of base 3420 sq. ft.

Soil bearing due to water = $\frac{4,020,000}{3420} = 1175 \text{ p.s.f.}$

Total soil bearing = 1071 + 1175 = 2246 p.s.f.

Allowable 3000 p.s.f.

Using a footing section as shown in drawing of prestressed tank
critical moment will occur in section marked (a)

Moment at point (a)

$$M = 3690 \times 4 = 14,760 \text{ in.}^2$$

Check for f_c .

$$f_c = \frac{2 \times 14,760}{0.429 \times 0.857 \times 10 \times 10 \times 12} = 65 \text{ psi}$$

$$\text{Allowable} = 250 \text{ psi.}$$

$$A_s = \frac{M}{f_{sjd}} = \frac{14,760}{18,000 \times .857 \times 10} = .096 \text{ sq. in./ft.}$$

Using $1/2'' \phi$ Area 0.30

$$\text{Spacing} = \frac{0.20}{.096} = 2.09 \text{ ft. USE } 12''$$

Check Shear at critical Section.

$$V = \frac{V}{bjd} = \frac{3690}{12 \times 0.857 \times 10} = 36.0 \text{ psi}$$

WIND STRESS:

Moment = 400,000 ft. # (from previous design)

$$I = \frac{\pi}{4} \left(\frac{32.67^4}{4} - \frac{32^4}{4} \right) = 47,200 \text{ ft. units}$$

$$f = \frac{400,000 \times 32.67}{47,200} = 277 \text{ #/ft. of wall}$$

Total soil bearing = 2525 p.s.f.

Therefore wind stress does not change dimensions of footing.

Amount of Part (a) = 100,000

100,000 x 10% = 10,000

Amount for Part (b)

$$100,000 \times 10\% = 10,000$$

Amount = 10,000

$$100,000 \times 10\% = 10,000$$

Value of Part (c) = 10,000

$$100,000 \times 10\% = 10,000$$

Amount of Part (d) = 10,000

$$100,000 \times 10\% = 10,000$$

Amount of Part (e)

Amount = 100,000 x 10% = 10,000

$$100,000 \times 10\% = 10,000$$

$$100,000 \times 10\% = 10,000$$

Amount of Part (f) = 10,000

Amount of Part (g) = 10,000

Amount of Part (h) = 10,000

Amount of Part (i) = 10,000

Amount of Part (j) = 10,000

Amount of Part (k) = 10,000

Amount of Part (l) = 10,000

DOM: DESIGN FOR COMPRESSION RING:

Refer to Figure 4 for general information.

Radius of sphere:

$$r^2 = 32 \times 32 + (r - 6)^2$$

$$r = 67.8 \text{ ft.}$$

Assume 5" thickness:

$$w_1 = 0.0625 + 0.030 = 0.0925 \text{ kips/sq. ft.}$$

$$\sin \phi_0 = \frac{5}{67.8} = 0.0442; \cos \phi_0 = 0.996$$

$$\sin \phi_1 = \frac{32}{67.8} = 0.472; \sin^2 \phi_1 = 0.222$$

$$\cos \phi_1 = 0.882$$

$$\phi_1 = 28^\circ$$

$$W_1 = 2 \pi \times 67.8 \times 67.8 \times 0.0925 (0.996 - 0.882) \\ = 320 \text{ kips.}$$

$$T_1 = \frac{320}{2 \pi \times 67.8 \times 0.222} = 3.39 \text{ kips/ft.} \\ = \frac{3390}{5 \times 12} = 56.5 \text{ p.s.i.}$$

$$H_1 = -3.39 + 0.0925 \times 67.8 \times 0.882 = 2.11 \text{ kips/ft.} \\ = 35.2 \text{ p.s.i.}$$

$$S_1 = \frac{330 \times 0.882}{2 \pi \times 0.471} = 95.5 \text{ kips. ring tension in edge member.}$$

$$S_0 = \frac{15 \times 0.996}{2 \pi \times 0.0442} = 53.7 \text{ kips compression in edge member} \\ \text{of lantern.}$$

EDGE MEMBER FOR DOM:

$$\text{Bearing on wall} = \frac{320,000}{201.5 \times 12 \times 8} = 16.6 \text{ p.s.i.}$$

$$\text{Allowable} = 750 \text{ p.s.i. in bearing.}$$

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Ring tension $\frac{1}{2}$ 95.5 kips; $A_s = \frac{95.5}{20} = 4.76$ sq. in. USE 6 - 1" ϕ .

Shear on 5" section; $v = \frac{320,000}{201.5 \times 12 \times 5} = 26.6$ p.s.i. (O.K.)

Temperature steel - Use 1/4" ϕ @ 12" both ways.

LARKIN COLUMN:

Compression in edge member = 53.7 kips

Allowable on concrete = 1360 p.s.i.

$A = \frac{53,700}{1360} = 39.5$ sq. in.

Use 6" x 7" coping.

that the total number of cases is 1000 and the number of cases in each age group is as follows:

(a) 1000 cases in total, 500 in each age group.

(b) 1000 cases in total, 250 in each age group.

(c) 1000 cases in total, 100 in each age group.

(d) 1000 cases in total, 50 in each age group.

(e) 1000 cases in total, 25 in each age group.

(f) 1000 cases in total, 10 in each age group.

(g) 1000 cases in total, 5 in each age group.

(h) 1000 cases in total, 2 in each age group.

(i) 1000 cases in total, 1 in each age group.

(j) 1000 cases in total, 0.5 in each age group.

(k) 1000 cases in total, 0.25 in each age group.

(l) 1000 cases in total, 0.1 in each age group.

(m) 1000 cases in total, 0.05 in each age group.

(n) 1000 cases in total, 0.025 in each age group.

(o) 1000 cases in total, 0.01 in each age group.

(p) 1000 cases in total, 0.005 in each age group.

(q) 1000 cases in total, 0.0025 in each age group.

(r) 1000 cases in total, 0.001 in each age group.

(s) 1000 cases in total, 0.0005 in each age group.

(t) 1000 cases in total, 0.00025 in each age group.

(u) 1000 cases in total, 0.0001 in each age group.

COMPARISON OF RELATIVE VALUES

As previously stated the comparison of the preceding designs will be based solely upon the quantity of materials (steel and concrete) used in each case, no attempt being made to evaluate fabrication costs and labor charges due to the many variable and unknown factors affecting the latter costs.

The total quantity of steel rods required in the case of the reinforced concrete tank is 82,277 pounds, and the corresponding total volume of concrete is 446 cubic yards. In the case of the prestressed concrete tank the total quantity of steel rods required is 27,734 pounds and the total volume of concrete is 318 cubic yards.

For these figures the difference in quantity of materials and the per cent saving gained by using prestressed construction are as follows:

	<u>R.C. Design</u>	<u>Prestressed Design</u>	<u>Difference</u>	<u>% Savings</u>
Total Steel	82,277 $\frac{1}{2}$	27,734 $\frac{1}{2}$	54,543 $\frac{1}{2}$	66.3%
Total Concrete	446 cu. yd.	318 cu. yd.	128 cu. yd.	29.6%

These figures include the additional saving gained by using the dome cover with the prestressed construction in preference to the flat slab cover of the reinforced concrete construction. The quantities of materials required for these covers alone (but including the center supporting column in the case of the flat slab) are:

	<u>R.C. Design</u>	<u>Prestressed Design</u>	<u>Difference</u>	<u>% Savings</u>
Total Steel	19,020 $\frac{1}{2}$	1,110 $\frac{1}{2}$	17,910 $\frac{1}{2}$	94%
Total Concrete	129 cu. yd.	51 cu. yd.	78 cu. yd.	60.5%

INVESTMENT IN THE UNITED STATES

The following figures are estimated for the calendar year 1914. The figures are based on the reports of the various departments of the Government, and are subject to revision. The figures are given in millions of dollars.

The total amount of investment in the United States in 1914 was \$1,100,000,000. This was an increase of \$100,000,000 over the total amount of investment in 1913. The increase was due to an increase in the amount of investment in the various departments of the Government, and to an increase in the amount of investment in the various industries of the country.

The following table shows the amount of investment in the various departments of the Government in 1914.

Department	Investment
War Department	\$1,000,000,000
Navy Department	\$100,000,000
Department of the Interior	\$50,000,000
Department of Agriculture	\$50,000,000
Department of Commerce	\$50,000,000
Department of Justice	\$50,000,000
Department of Education	\$50,000,000
Department of Labor	\$50,000,000
Department of Health	\$50,000,000
Department of Social Welfare	\$50,000,000
Department of Public Health	\$50,000,000
Department of Mental Hygiene	\$50,000,000
Department of Physical Education	\$50,000,000
Department of Recreation	\$50,000,000
Department of Art	\$50,000,000
Department of Music	\$50,000,000
Department of Literature	\$50,000,000
Department of Science	\$50,000,000
Department of Technology	\$50,000,000
Department of Engineering	\$50,000,000
Department of Architecture	\$50,000,000
Department of Fine Arts	\$50,000,000
Department of Music	\$50,000,000
Department of Literature	\$50,000,000
Department of Science	\$50,000,000
Department of Technology	\$50,000,000
Department of Engineering	\$50,000,000
Department of Architecture	\$50,000,000
Department of Fine Arts	\$50,000,000

The following table shows the amount of investment in the various industries of the country in 1914.

Industry	Investment
Manufacturing	\$1,000,000,000
Transportation	\$100,000,000
Commerce	\$50,000,000
Finance	\$50,000,000
Insurance	\$50,000,000
Real Estate	\$50,000,000
Public Utilities	\$50,000,000
Health	\$50,000,000
Education	\$50,000,000
Recreation	\$50,000,000
Art	\$50,000,000
Music	\$50,000,000
Literature	\$50,000,000
Science	\$50,000,000
Technology	\$50,000,000
Engineering	\$50,000,000
Architecture	\$50,000,000
Fine Arts	\$50,000,000
Music	\$50,000,000
Literature	\$50,000,000
Science	\$50,000,000
Technology	\$50,000,000
Engineering	\$50,000,000
Architecture	\$50,000,000
Fine Arts	\$50,000,000

Then, assuming the same cover (flat slab) in both designs, the quantity of materials in each case would represent a true comparison of the relative economy of materials for the two types of construction. These figures are:

	<u>R.C. Design</u>	<u>Prestressed Design</u>	<u>Difference</u>	<u>Savings</u>
Total Steel	82,377 #	45,644 #	36,633 #	44.5%
Total Concrete	446 cu. yd.	396 cu. yd.	50 cu. yd.	11.2%

These members are not only (1) to be kept in the
position of affairs in each case and to be kept in the
of the affairs of the country in the case of the

These members are

The following members are			
1917	1918	1919	1920
1921	1922	1923	1924

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The following members are			
1925	1926	1927	1928
1929	1930	1931	1932

The following members are
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The following members are
The following members are
The following members are

The following members are			
1933	1934	1935	1936
1937	1938	1939	1940

CONCLUSION

An indication of the advantages to be gained by using prestressed concrete construction is given by the figures representing the quantity of materials required for this type of construction compared to the quantity required for conventional construction. It is realized that a comparison made solely on this basis is not indicative of the overall costs in each case, but a complete study of this matter is beyond the scope of this thesis as actual construction costs were not available to the authors.

On the other hand, the figures presented for comparison do not reveal all of the advantages gained by using prestressed construction in this case. For the prestressed tank cracklessness is guaranteed and this quality is, of course, of primary importance in a pressure vessel. To obtain a comparable degree of security in this respect in the case of the reinforced concrete tank it was considered advisable to reduce the allowable tensile stress in the circumferential steel to 12,000 p.s.i. or about 67 per cent of the usual value for tensile reinforcement. For the corresponding steel in the prestressed tank it was possible to utilize the full allowable stress, higher strength steel being used economically in this case.

It should also be pointed out that the lower total weight of the structure and hence the lower soil bearing pressures (for the same size footings) might be the controlling factor in a location of low soil bearing.

Introduction

The purpose of this book is to provide a comprehensive overview of the current state of research in the field of artificial intelligence. It is intended for researchers, students, and practitioners who are interested in the latest developments in this rapidly evolving field. The book covers a wide range of topics, including machine learning, natural language processing, computer vision, and robotics. It also discusses the ethical implications of AI and the challenges that lie ahead. The book is written in a clear and concise style, making it accessible to a broad audience. It is hoped that this book will serve as a valuable resource for anyone looking to stay up-to-date on the latest in AI research.

In the first chapter, we introduce the field of artificial intelligence and discuss its history and current state. We then move on to a detailed discussion of machine learning, which is one of the most active areas of research in AI. This chapter covers the basics of machine learning, as well as more advanced topics such as deep learning and reinforcement learning. The next chapter focuses on natural language processing, which is the study of how computers can understand and generate human language. This chapter covers a wide range of topics, including text classification, sentiment analysis, and machine translation. The following chapter discusses computer vision, which is the study of how computers can interpret visual information from the world. This chapter covers topics such as image classification, object detection, and facial recognition. The final chapter discusses the ethical implications of AI and the challenges that lie ahead. We discuss issues such as privacy, security, and the potential for AI to be used for harmful purposes. We also discuss the need for responsible AI development and the importance of ensuring that AI is used for the benefit of humanity.

It is hoped that this book will provide a valuable overview of the current state of research in AI and serve as a useful resource for researchers, students, and practitioners alike. The book is written in a clear and concise style, making it accessible to a broad audience. It is hoped that this book will serve as a valuable resource for anyone looking to stay up-to-date on the latest in AI research.

The free base which is well suited to the prestressed construction obviates the necessity of providing for moment in the circumferential footing at the base of the wall.

These factors would, in some cases, counter-balance the additional cost of construction for a prestressed tank and thus lend more reality to the figures presented for comparison.

In the process of design of the prestressed tank the authors attempted to design a flat slab cover using prestressed beam theory but this was found to be impractical from a construction standpoint due to the requirement of a constant percentage of steel throughout the cover to maintain economy of design. The difficulty encountered in carrying out this requirement would present itself in the fabrication stage. It appears to be impractical to cut off any of the rods at the interior of the span of the slab, to anchor these rods at the interior point, and to apply the necessary force for prestressing either before or after setting of the concrete. Further study of this subject will be required to devise a means of overcoming these difficulties in order that the prestressing theories may be applied to members whose cross sectional area varies with length.

THE FIRST PART OF THE REPORT IS A SUMMARY OF THE WORK DONE

IN THE YEAR 1914. THE SECOND PART IS A SUMMARY OF THE WORK DONE
IN THE YEAR 1915. THE THIRD PART IS A SUMMARY OF THE WORK DONE

IN THE YEAR 1916. THE FOURTH PART IS A SUMMARY OF THE WORK DONE
IN THE YEAR 1917. THE FIFTH PART IS A SUMMARY OF THE WORK DONE
IN THE YEAR 1918. THE SIXTH PART IS A SUMMARY OF THE WORK DONE

IN THE YEAR 1919. THE SEVENTH PART IS A SUMMARY OF THE WORK DONE
IN THE YEAR 1920. THE EIGHTH PART IS A SUMMARY OF THE WORK DONE
IN THE YEAR 1921. THE NINTH PART IS A SUMMARY OF THE WORK DONE
IN THE YEAR 1922. THE TENTH PART IS A SUMMARY OF THE WORK DONE

IN THE YEAR 1923. THE ELEVENTH PART IS A SUMMARY OF THE WORK DONE
IN THE YEAR 1924. THE TWELFTH PART IS A SUMMARY OF THE WORK DONE
IN THE YEAR 1925. THE THIRTEENTH PART IS A SUMMARY OF THE WORK DONE
IN THE YEAR 1926. THE FOURTEENTH PART IS A SUMMARY OF THE WORK DONE
IN THE YEAR 1927. THE FIFTEENTH PART IS A SUMMARY OF THE WORK DONE

IN THE YEAR 1928. THE SIXTEENTH PART IS A SUMMARY OF THE WORK DONE

IN THE YEAR 1929. THE SEVENTEENTH PART IS A SUMMARY OF THE WORK DONE

IN THE YEAR 1930. THE EIGHTEENTH PART IS A SUMMARY OF THE WORK DONE

IN THE YEAR 1931. THE NINETEENTH PART IS A SUMMARY OF THE WORK DONE

IN THE YEAR 1932. THE TWENTIETH PART IS A SUMMARY OF THE WORK DONE

IN THE YEAR 1933. THE TWENTY-FIRST PART IS A SUMMARY OF THE WORK DONE

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APPENDIX

THE HISTORY OF THE UNITED STATES OF AMERICA

FROM THE FIRST SETTLEMENTS TO THE PRESENT TIME

BY JAMES OSGOOD, ESQ.

LONDON: 1834.

Printed by J. JOHNSON, in Pall-mall.

1834.

Vol. I. Part I. The first settlement.

1607-1619.

Vol. I. Part II. The second settlement.

1620-1639.

Vol. I. Part III. The third settlement.

1640-1649.

Vol. I. Part IV. The fourth settlement.

1650-1659.

Vol. I. Part V. The fifth settlement.

1660-1669.

Vol. I. Part VI. The sixth settlement.

1670-1679.

1680-1689.

Vol. I. Part VII. The seventh settlement.

1690-1699.

Vol. I. Part VIII. The eighth settlement.

1700-1709.

Vol. I. Part IX. The ninth settlement.

1710-1719.

BIBLIOGRAPHY - Continued

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1. The above information was obtained from a confidential source.

Source: [redacted]

2. The above information was obtained from a confidential source.

Source: [redacted]

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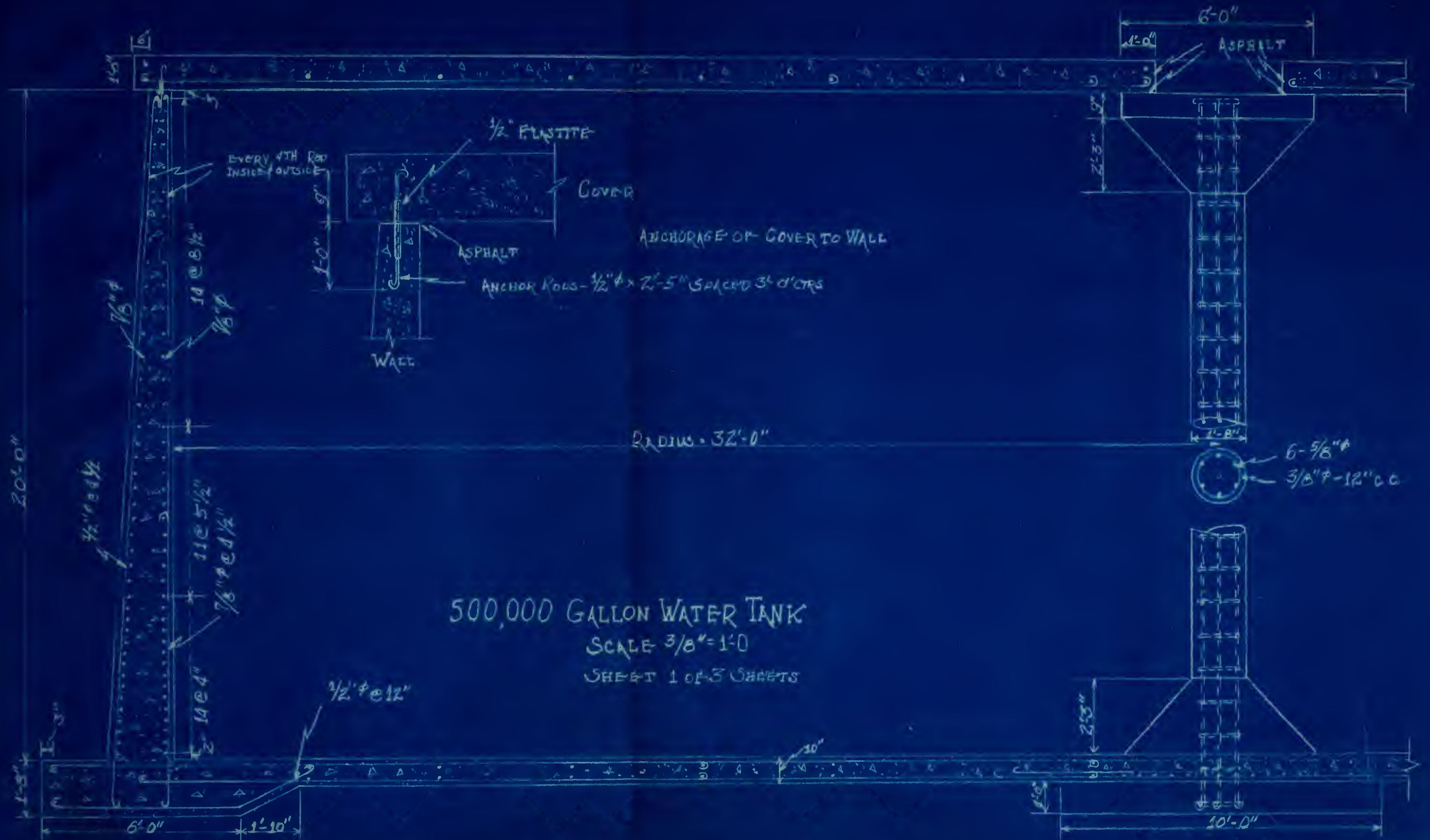
21. The above information was obtained from a confidential source.

22. The above information was obtained from a confidential source.

23. The above information was obtained from a confidential source.

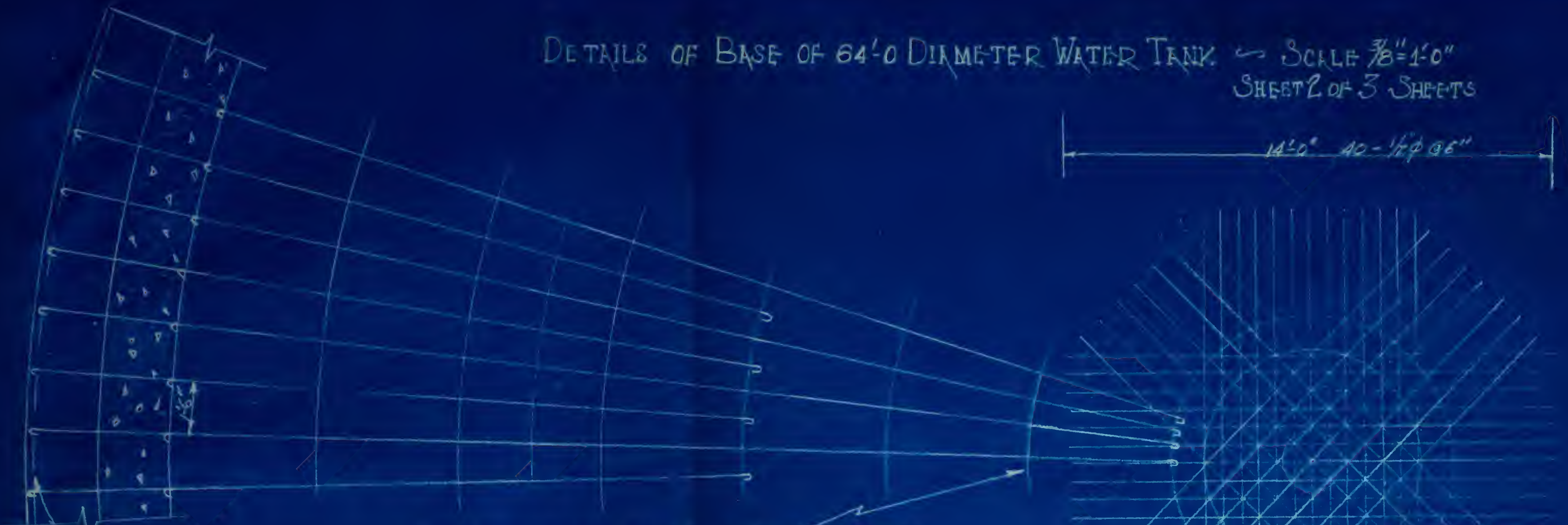
24. The above information was obtained from a confidential source.

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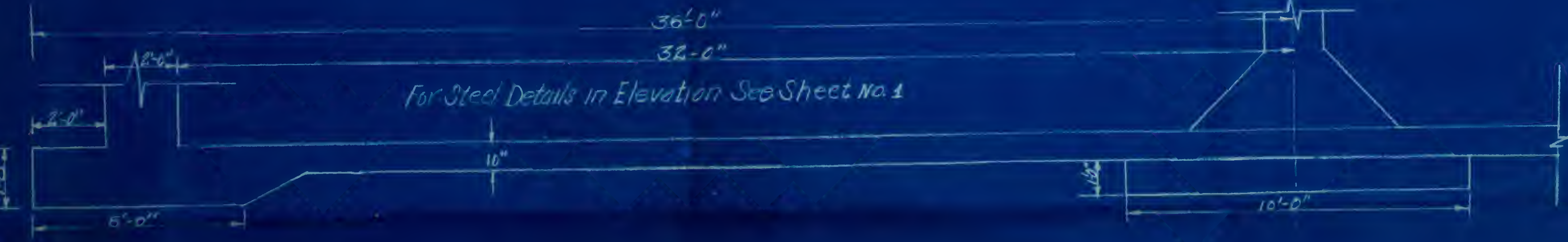




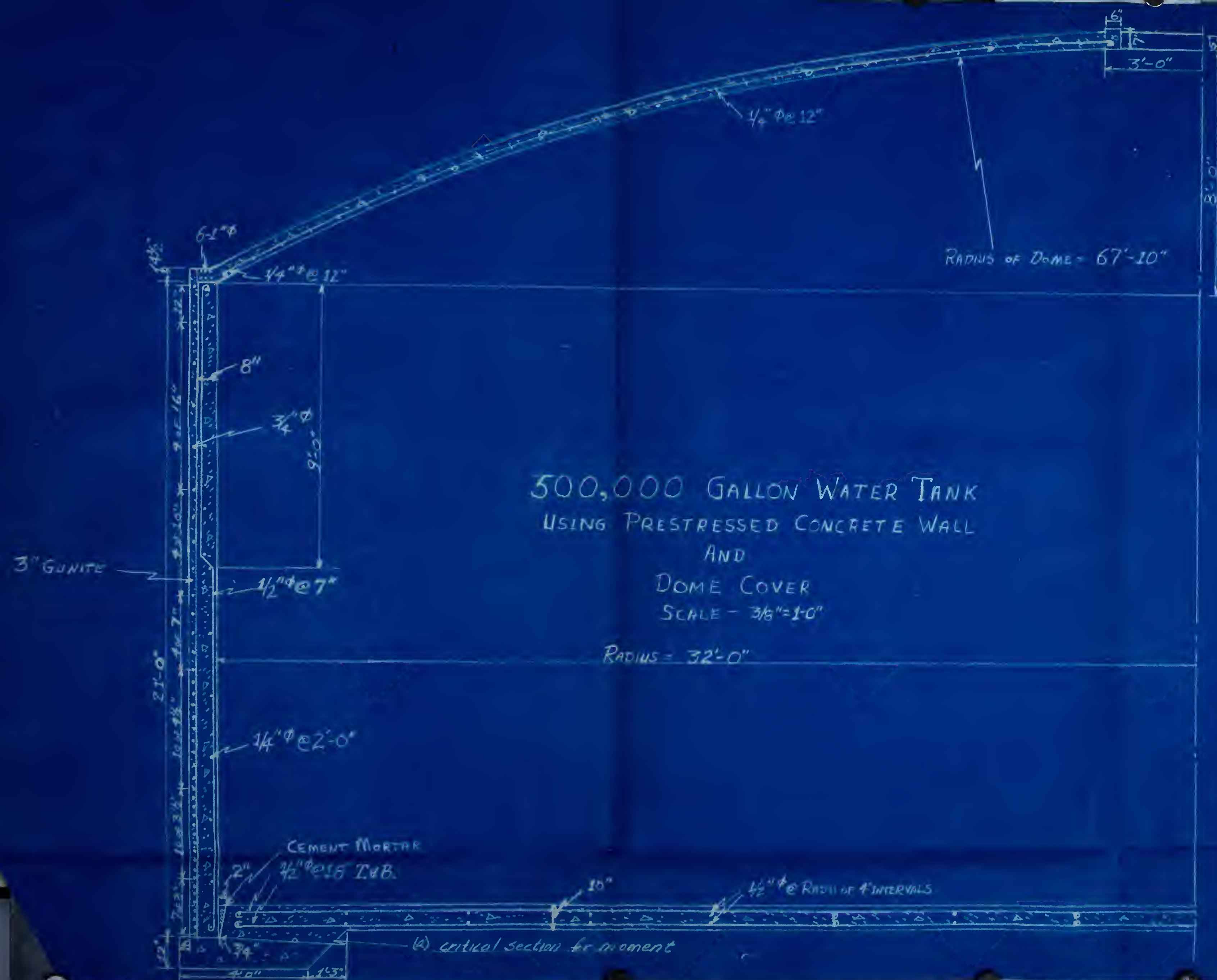
DETAILS OF BASE OF 64'-0 DIAMETER WATER TANK. → SCALE $\frac{3}{8}"=1'-0"$ SHEET 2 OF 3 SHEETS



2- $\frac{3}{4}"$ Top & Bottom
 8- $\frac{1}{2}"$ Spaced radially at 4'-0 = 32'-0 Top & Bottom
 Note: All bends shown are standard



For Steel Details in Elevation See Sheet No. 1



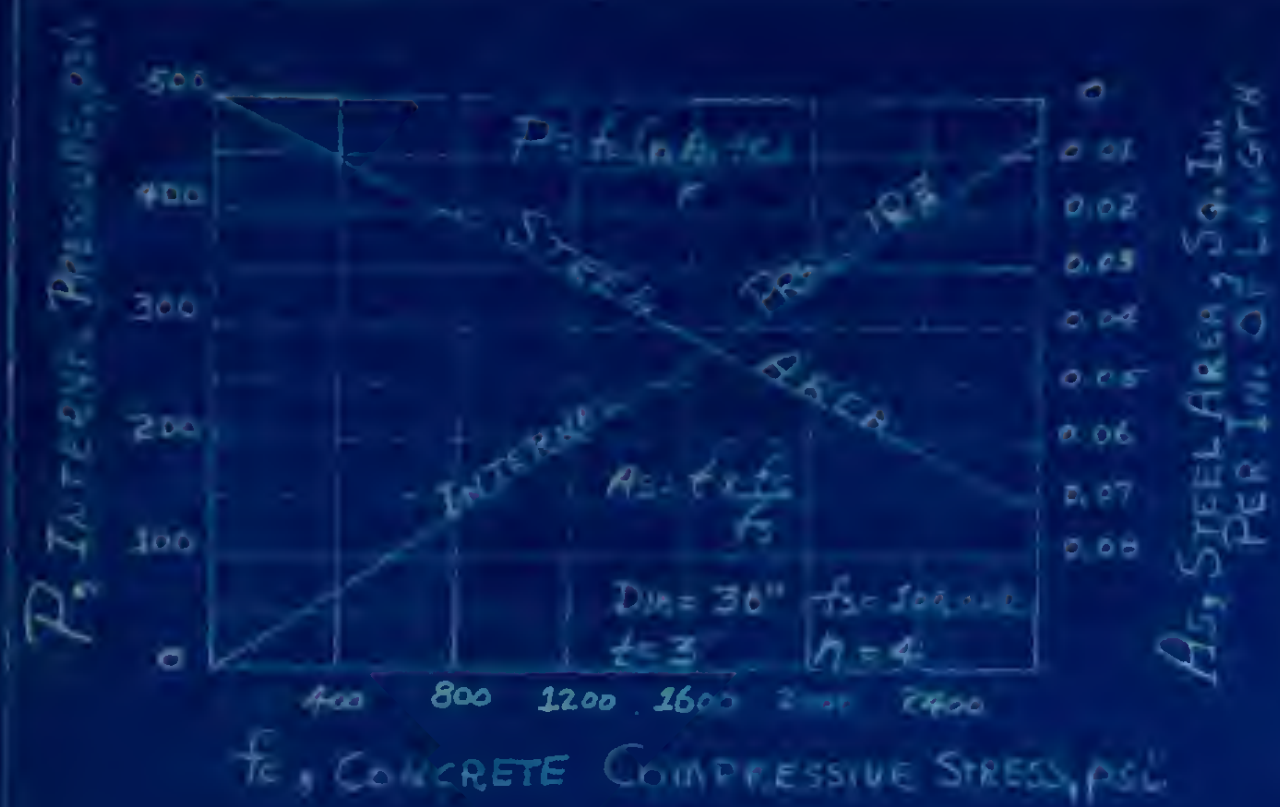


FIG. 1

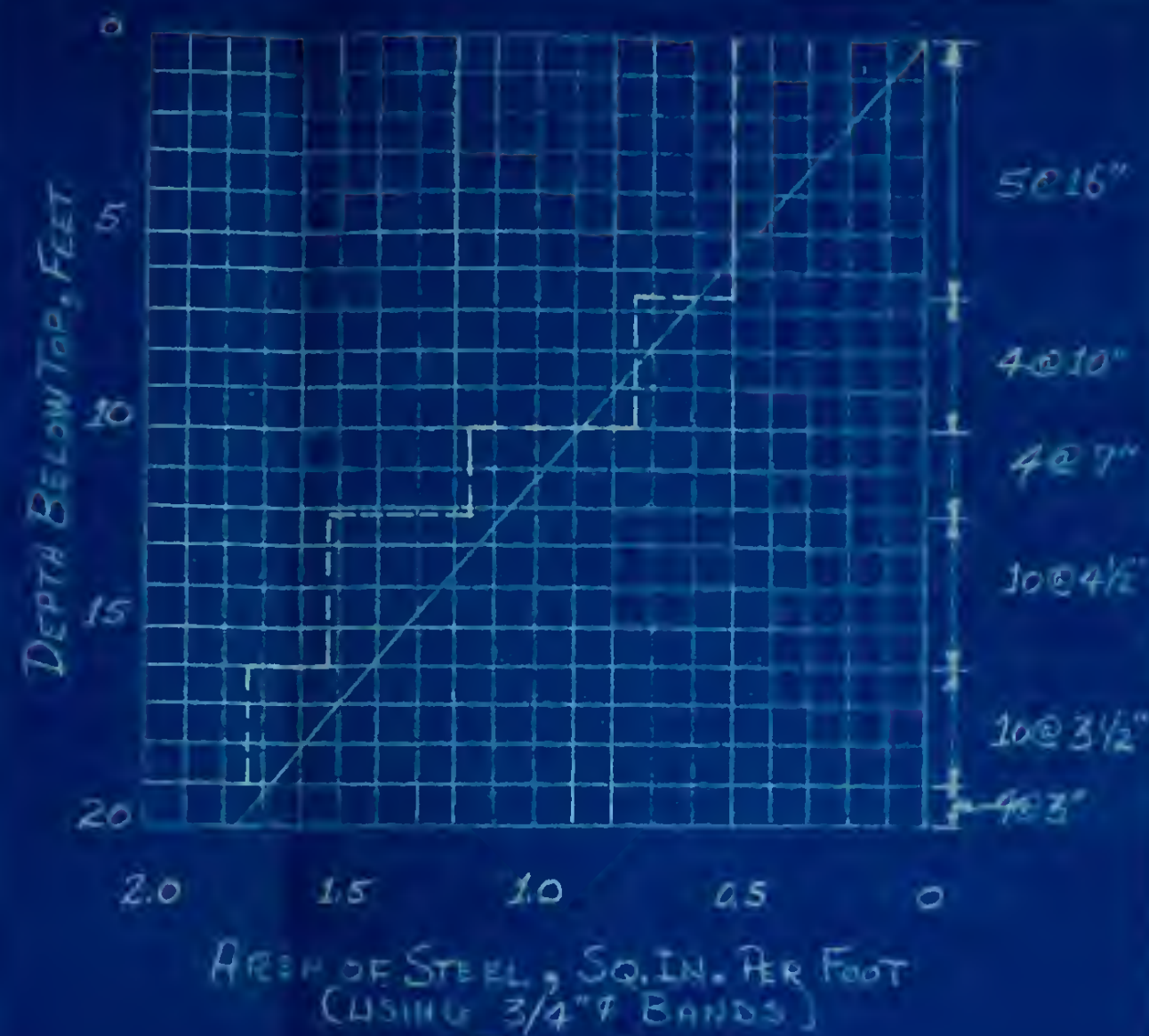


FIG. 2

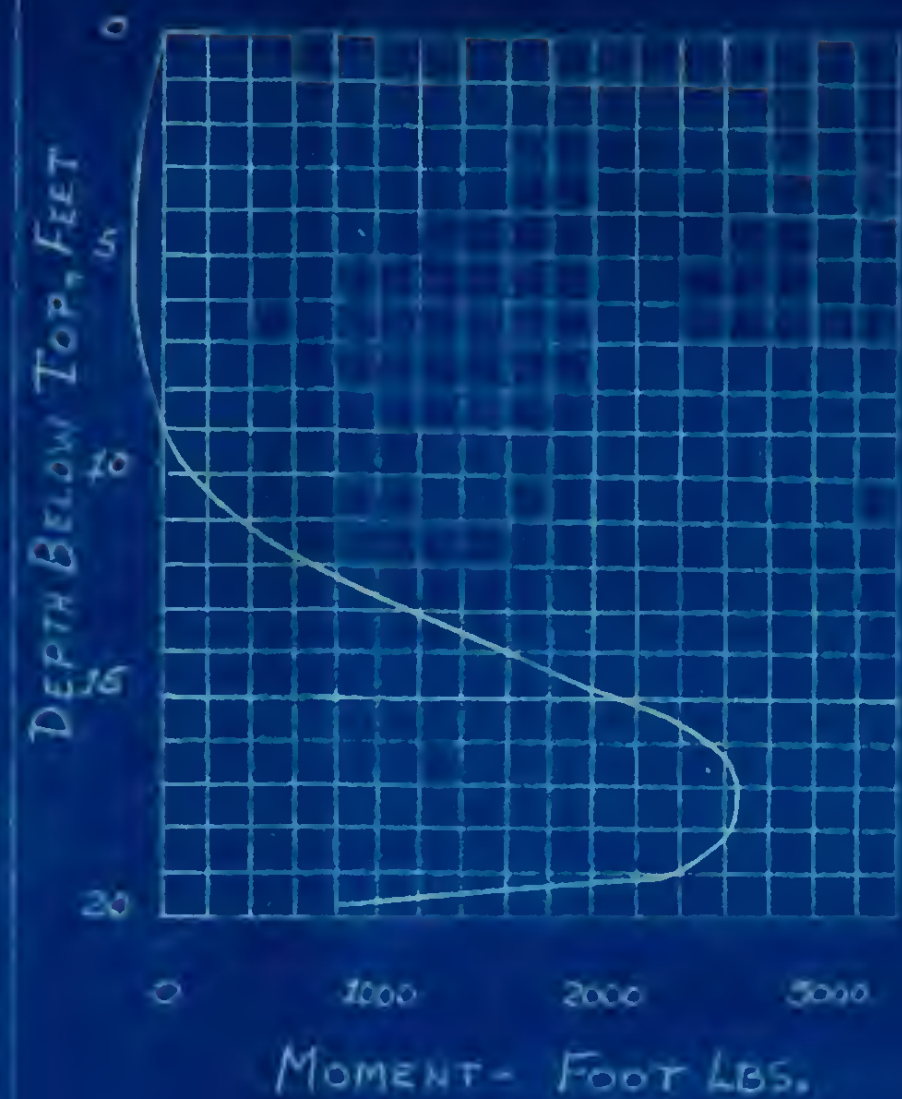


FIG. 3



FIG. 4

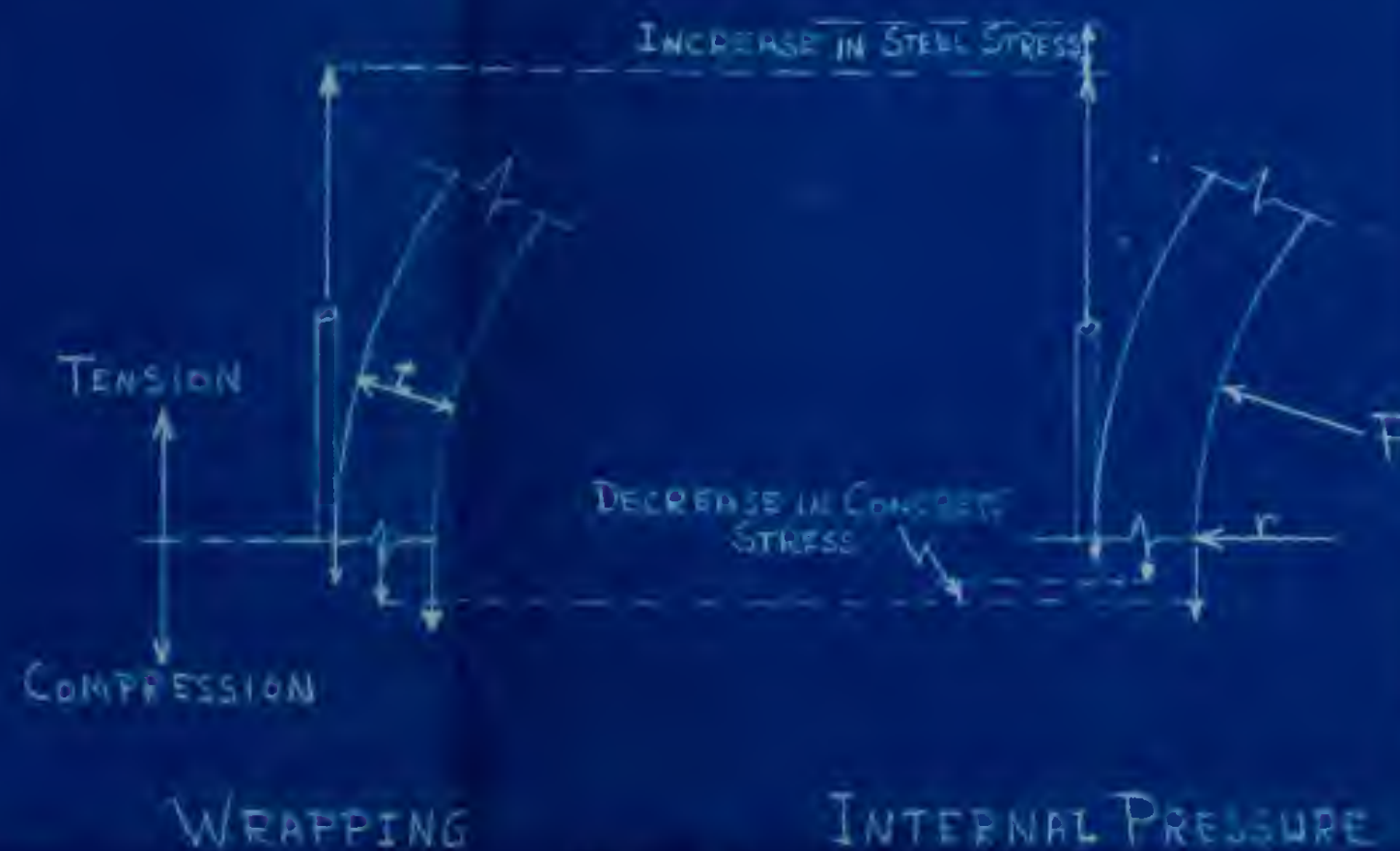


FIG. 5

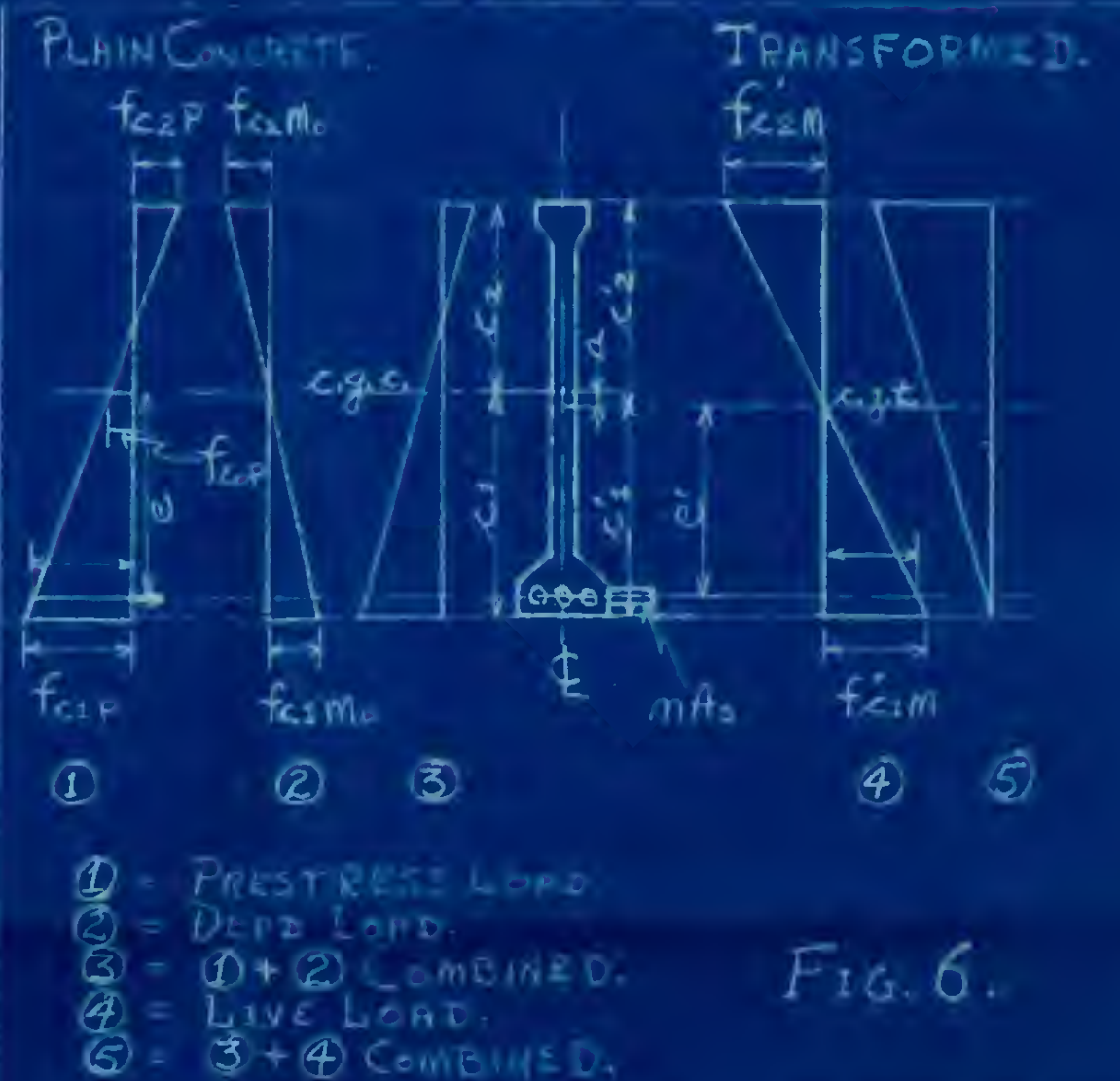


FIG. 6

thesZ3

An investigation of prestressed concrete



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